WATERWAY ENGINEERING

A TEXT AND HANDBOOK TREATING OF THE DESIGN, CONSTRUCTION, AND MAINTENANCE OF NAVIGABLE WATERWAYS

BY

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THE TECHNOLOGY PRESS
MASSACHUSETTS INSTITUTE OF TECHNOLOGY
CAMBRIDGE
1936

First Published in Germany in 1927
BY JULIUS SPRINGER, BERLIN
under the title
DER VERKEHRSWASSERBAU

PRINTED IN THE UNITED STATES OF AMERICA
BY THE MURRAY PRINTING COMPANY
CAMBRIDGE, MASS.

TRANSLATOR'S PREFACE

ALTHOUGH no political barriers are recognized in the engineering sciences, there are real or fancied obstacles which prevent the dissemination of knowledge to the busy designing engineer. One of the common barriers involves the differences in language. Every generation finds each outstanding civilized nation producing a number of individuals who might be called geniuses; and for the most part, the thought of these men is directed along the great scientific and social problems of the nation. Hence, the genius of one country may follow along entirely different lines of endeavor from that of another. America's expansive territory and its business enterprise made paramount the development of transportation and communication, resulting in a communication network far surpassing all others of the world. Until quite recently, however, this country has been comparatively little concerned with river regulation problems and in providing transportation facilities over its huge unimproved system of natural waterways; their rectification and regulation were considered unnecessary. The public now recognizes a great natural resource in its undeveloped system of water routes. As an indirect consequence thereof, many ingenious methods of regulation have of late been developed in America. This country is now on the threshold of what promises to be the greatest era of river regulation in history.

Europe, because of the density of its population, was forced to consider the matter of river regulation seriously several decades ago. Thus Germany began a systematic scheme of river improvement about the middle of the past century and has all but completed this work; she is now concerned chiefly with the maintenance of river control structures and in providing artificial connecting canals to link together the already improved natural system of waterways. It is only reasonable, therefore, to presume that a definite technique of river regulation has been evolved and that the methods used abroad are bound to include valuable suggestions for the American engineer. There is no intention or desire on the part of the writer to transplant German methods of river regulation in America, but rather to disseminate some of the knowledge of over a century's accumulation in a country where the improvement of waterways has been a paramount issue.

John R. Freeman, a foremost American hydraulic engineer, in consequence of his broad acquaintance and frequent contact with foreign

engineers, recognized the great value that might accrue to the profession by an exchange of thought. In his visits abroad he found that the application of experimental hydraulics to practical design had produced astounding results in facilitating and bettering the designs of hydraulic structures. With the desire of effecting this exchange of thought of Americans with foreign hydraulic engineers, he presented funds to three leading engineering societies (American Society of Civil Engineers, American Society of Mechanical Engineers, and Boston Society of Civil Engineers), the interest on which is to provide young hydraulic engineers with scholarship stipends for study and research abroad, and foster the diffusion of foreign knowledge of hydraulics in this country. Besides these funds, Dr. Freeman provided additional financial support to accelerate the realization of his plan of returning something for the advancement of the profession (hydraulic engineering) of which he was a member. The translation of this treatise from the German was thus conceived by Mr. Freeman and undertaken by the writer with the hope of bringing to light otherwise more or less inaccessible information concerning foreign practice in the design, construction, and maintenance of navigable waterways.

There are a number of excellent German books on applied hydraulics; this one was chosen largely because of its recent date of publication and the scope of its subject matter. It is to be hoped that other notable foreign writings in hydraulics will be published in English so as to be readily accessible to American hydraulic engineers.

An earnest effort has been made in the translation to retain the original viewpoint of the author. As is to be expected in any subject which borders on both art and science, diversities of opinion are bound to exist. All German hydraulic engineers do not agree with the author in all matters presented in this book; nor is it to be expected that all Americans in the profession will agree with him verbatim. This should not detract from the treatise; it is simply inherent in the subject. In places where it was deemed especially advisable the writer has provided explanatory notes calling attention to characteristic individualities.

Although the entire book of over eight hundred pages was translated into English and edited, after careful consideration of the subject matter it was decided to print only those parts which provide information not readily accessible to American engineers. In this way it was possible to present the salient information at a reasonable cost of publication, and lower selling price of the book, and at the same time avoid further duplication of material available in American treatises. The omissions in all cases consisted of entire chapters, including Parts I, II, VII, VIII, and XI. Part I of the German edition, entitled "Shipping — Its Nature

and Its Significance," contains nothing new or unavailable to American engineers. Part II, entitled "Water," treats of the general field of hydrology — a subject that already has been well developed in such books as "Hydrology" by D. W. Mead, "Elements of Hydrology" by A. F. Meyer, and "Stream Gaging" by W. A. Liddell. Part VII, "Dams," contains material which has been well treated in "The Design and Construction of Dams" by Edward Wegman. The subject matter of Part VIII, "Water Power Plants," is covered quite thoroughly by "Water Power Engineering" by D. W. Mead, "Water Power Engineering" by H. K. Barrows, and similar treatises. Part XI, entitled "Ports," is well treated by such books as "Port Development" and "Ports and Terminal Facilities," both by R. S. MacElwee.

The elimination of these chapters from the English edition does not greatly affect the unity or coherence of the subject matter inasmuch as virtually nothing has been omitted from the original treatment which concerns the construction and improvement of navigable waterways, with the exception of the development of port facilities. The latter forms an independent subject in itself.

The writer wishes to express his gratitude to all who have had a part in assisting with the presentation of this book in the English. He is especially indebted to the late Dr. John R. Freeman who, as founder of a group of fellowships for the study of hydraulic practice abroad, made it possible for the writer to spend two years in Europe where he could become familiar with foreign practice and German technical terms in hydraulic engineering, thus facilitating translation of the treatise; also for Mr. Freeman's complete financial support and sustained personal interest in this undertaking. The writer also wishes to express his appreciation to Mr. J. Rhyne Killian, Jr., Editor of the *Technology Review,* for his work in reviewing the entire treatise and for his valuable suggestions, constructive criticism, and close coöperation with the writer in the production of the English edition of this book.

L. G. S.

Minneapolis, Minn.

AUTHOR'S PREFACE

THIS treatise is intended to provide the designing and construction engineer with a condensed presentation of present-day knowledge of waterway engineering. It is not a book on transportation but rather one on that phase of hydraulics which serves transportation. The book is intended to be scientific, not in the sense that all theories are presented, but rather in the sense that the plausibility of various statements is critically analyzed. Antiquated methods have not been entirely disregarded because, according to experience, the antiquated methods of today may frequently be re-evaluated by ingenious changes. The older procedures, however, have been treated very lightly and in some cases have only been mentioned. The sources of information from which further data may be obtained have been indicated by footnotes. Inasmuch as the treatise is intended particularly for practical purposes, the development of the theories has been confined to a minimum space. It seemed more useful to present the theory in a condensed but well-analyzed form than to give extended developments thereof. In many cases new presentations or conceptions have been discussed. Thus, for example, the tables of water velocities have been presented in a different way than has been customary heretofore; methods of computation for shore walls, locks, lock gates, etc., were presented in a newer form.

The treatise is limited to the subject matter of waterway engineering, a profession which I have pursued in practice for twenty-five years and as a teacher for thirteen years. The treatment of questions of land reclamation as related to hydraulics has been avoided as far as possible. Discussion of the occurrence and movement of water has been condensed, the necessary practical information being emphasized. The book is not intended as a treatise for advanced training, but rather as an aid to the scientifically minded, practical engineer. In order to save space and maintain a systematic presentation, it was necessary to treat the more theoretical questions as a group; thus, for example, all questions concerning the occurrence and movement of water, regardless of whether river or ocean was concerned, were treated together. Locks of all types, canals, harbors, etc., have been discussed in individual sections, but have not been segregated according to divisions of river works, sea works, etc.

In the presentation of this treatise I especially acknowledge the valuable coöperation of my former assistant, Dr.-Ing. F. Collorio, and my

present assistant, Dipl.-Ing. Heinze. The former prepared most of the drawings, numerical tables, part of the data; the latter read proof with particular care. Furthermore, my assistant, Dr.-Ing. A. Streck, contributed materially to the book. All of these men have also aided greatly by a critical examination of the subject matter. Many firms and engineers have provided me with valuable drawings and discussions. Thanks are again expressed to all of these at this time. I want to especially acknowledge the coöperation of the publisher [of the German edition], Julius Springer, and thank him for his fine make-up of this book.

O. Franzius.

Hanover, September 1927.

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ABBREVIATIONS FOR PERIODICALS

Bauing.	*Der Bauingenieur*
Baut.	*Bautechnik*
Beton Eisen.	*Beton und Eisen*
Dt. Bauzg.	*Deutsche Bauzeitung*
D. W. W.	*Deutsche Wasserwirtschaft*
ETZ.	*Elektrotechnische Zeitschrift*
Ö. W.	*Österreichische Wochenschrift für den öffentlichen Baudienst, Wien*
Schweiz. Bauzg.	*Schweizerische Bauzeitung, Zurich*
Z. Bauw.	*Zeitschrift für Bauwesen, Berlin*
Z. Binnensch.	*Zeitschrift für Binnenschiffahrt*
Zentralbl. Bauverw. . . .	*Zentralblatt der Bauverwaltung*
Z. ö. I. A. V.	*Zeitschrift des österreichischen Ingenieur- und Architekten Vereins, Wien*
Z. V. d. I.	*Zeitschrift des Vereins deutscher Ingenieure, Berlin*

NOTE ON THE ILLUSTRATIONS

For economy, the illustrations in this book were reproduced from the German edition and consequently show original German signs, terms, and legends. These have been translated only where their meaning might not be apparent to American readers.

The use of the German illustrations explains why the decimal point, when it appears in numbers shown on illustrations, is designated by the German convention (similar to our comma), rather than by the dot or period sign used by draftsmen in English-speaking countries.

It should further be noted that the German abbreviation *N. W.* is equivalent to our LW (low water).

WATERWAY ENGINEERING

PART ONE — RIVER CONTROL

A. GENERAL

TWO entirely different problems must be dealt with in river control. One consists in attaining a good discharge, in which detritus movement is of interest only insofar as it affects the water movement. This condition must be brought about in cases where the combined interests of agriculture, power development, and city planning are of prime importance, but where large depth for navigation is unimportant.

The second problem is that of developing the greatest possible uniform depth in rivers, and necessitates a well-regulated detritus movement especially at LW and MW stages. For this purpose the scouring capacity of the river is extremely important, the movement of the water being of interest only insofar as it is the cause of scouring capacity. We are not concerned with a uniform movement of water but with equalization of detritus movement. Because of the lack of uniformity in various river sections, this *is possible only with variation of water movement*, and, therefore, with corresponding variation in the gradient. The development of uniform detritus movement is particularly important in streams where depth is the chief characteristic of the river; that is, in navigable streams.

Every natural river forms bends. Erosion takes place at the concave side of the bend. At the convex side, sand banks form because of the low scouring force. Submerged bar formations occur in the transitions. There is a medium scouring force at transitions, but it is usually too small to keep the channel deep enough for navigation. In order to effect uniform transportation of sediment the shoals must be deepened at the cost of the depth of the pools; that is, the depth must be made uniform. *The best solution is not obtained by making the gradient uniform, as is frequently still attempted, but by balancing the available scouring forces.* One adjustment, however, must be made in the water movement. This concerns the distribution of velocity in bends. The velocity must be lessened in the deep part of the pool and increased in the remainder of the section.

It can be demonstrated that the transporting force corresponds to the expression[1] $S = atJ$. The scouring force possesses a similar value. In spite of the fact that especially great depths occur at the pools, nature has taken care that the value of S does not become too large, the gradients at bends always being flatter than at transitions. Nevertheless, because of the great depths at bends, the scouring force there is usually much greater than at transitions. If the depth at the transitions could be mechanically increased, the scouring capacity would be augmented, but such a procedure would be of no avail since subsequent HW would again leave shoals. Furthermore, rapid improvement for navigation cannot be accomplished by dredging the sills.

Even good dredging does not alter the requirement that the river must be forced to scour out the shoals of its own accord. This is best accomplished by increasing the gradient over the shoals. Even a small diminution of the gradient in the pools, which are very long, makes possible a substantial increase in gradient at the transitions, because the steep gradient over the shoal begins only a comparatively short distance above the actual shoal crest, just as in the case of weirs. Toward the upstream side, the level of the backwater is affected far up the bend; downstream the steep gradient quickly adjusts itself to the flatter gradient of the following bend. Stretches having a flat gradient are thus very long, while the steep stretches over the shoals are comparatively short. In large rivers with excessive width, the width of the transitions should be narrowed down more substantially than has been done heretofore.

Just as detritus movement is dependent upon the kinetic energy of the water $mv^2/2$, resistance to navigation is likewise dependent upon this value. Doubtless an increase in velocity at individual short stretches causes very slight inconvenience which is acceptable to steamship operators if the usable depth of the stream at LW stages can thereby be increased. It is not even necessary to increase the power of the towboats to cope with the increase in gradient at the shoals. The towboats will simply travel somewhat slower at these locations in the case of properly regulated streams, but will be able to travel more rapidly in the bends because here the velocity of flow will be diminished. The total consumption of towing energy for propelling the ship, to be sure, will be slightly raised because of the increased irregularity in velocity of the water; the value of the stream for transportation, on the other hand, will become much greater since considerably heavier loading can be handled than could be otherwise.

[1] In this equation, when expressed in English units, S is the transporting force in pounds per square foot, J is the slope of the water surface, t the depth of the stream in feet, and a the weight of water in pounds per cubic foot.

After the fundamental principles of river regulation have been grasped, crude mistakes in carrying out such work will not be made. Individual mistakes may be made, but the form of the river as a whole will be continually improved.

It is of particular importance in river construction to recognize that water does not move forward in uniform, parallel layers but is subject to regular pulsations combined with very irregular eddy and roller formations. The velocity varies continually. Mathematically, every change in velocity must correspond to a change in the surface configuration.

In case of a rectilinear stretch in which the section is of a trough form, the water flows most rapidly in the middle and slowest at the shores. Because of friction of water against water, the flow is attracted from the shores toward the center and continually moves from the slow edge-velocities to the more rapid center-velocity. A transverse gradient occurs from both sides toward the center, and the water moves transversely toward the center. The surface of the center of the river must lie deeper than the edges.

This phenomenon occurs not only in the case of a constant quantity of flow, but it is most pronounced when the water stage sinks. When the water rises, the condition is reversed. The phenomenon of ships being forced off their courses during the stage of rising water is significant proof of this condition. In consequence of this flow, characteristic transverse currents develop even in straight river stretches, and eddies are generated, causing erosion of the shore (Fig. 1). Since a homogeneous river bottom practically never exists, this theoretically uniform

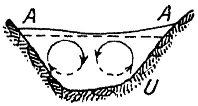


Fig. 1. Transverse currents in a regular cross-section

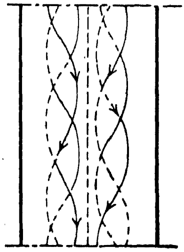


Fig. 2. Development of an irregular cross-section

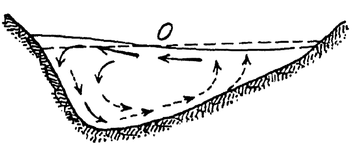


Fig. 3. Helical movement of water

section rapidly takes a form similar to Fig. 3, which is the beginning of a bend. The transverse movement then becomes greater because of the formation of the bend. Furthermore, the inertia of the water flowing through a sharp bend tends toward a tangential rectilinear direction and is heaped on the concave shore causing a one-sided transverse

gradient. The superelevation of the water at the surface corresponds to the velocity in the curve. The deeper from the surface, the lower the velocity and the less is the piling action (due to centrifugal force). Therefore, the water is not normally in static equilibrium to the direction of flow; near the concave shore there is an overpressure in the upper area as compared to the lower regions. It follows that water must continually flow from top to bottom at the concave shore (Fig. 3). Hence transverse eddies increase in magnitude at bends in rivers. These eddies consume energy and thereby decrease the velocity, provided this increased consumption of energy is not compensated by a steepening of the gradient[1] (formation of unsteadiness in the gradient). Besides consuming energy, the transverse eddies tear down the concave shore, especially during HW stages. Underwashing of the shore often takes place at binding (loamy) types of ground. The shore may be held erect by the water pressure during the period of HW, but it falls as the counter pressure is removed when the water level sinks. Similar action occurs in sandy ground. Even more serious developments arise in this case because the sand becomes saturated with water. This water flows toward the river during LW and causes caving of the bank.

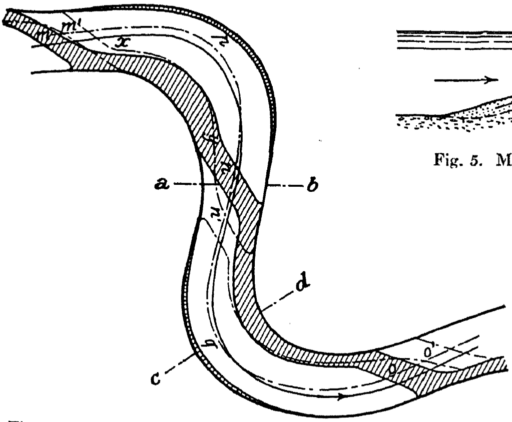


Fig. 4. Condition of the movable bed in a river which transports detritus (bed sediment)

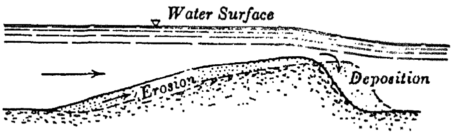


Fig. 5. Mode of progression of a gravel bar in the upper Rhine

After a bend has once started, there is a tendency for the curvature to become constantly larger, forming a sharper loop. The deposits of washed-out material at the end of the bend aid this phenomenon.

Uninterrupted detritus movement usually occurs only at HW and in this case the detritus follows the shortest path. According to important experiments by Engels, the particles move from one convex

[1] It is improbable, as is often assumed, that the water in these transverse eddies leaves the curve at a lower velocity than it enters the curve. This is due to enlargement of the section at the end of every bend. It is very probable that the rate of fall in such a curve is such that the average velocity remains uniform. An exact knowledge of this occurrence may be obtained only through experiment.

shore to another, the shoals (transitions) between two bends being crossed diagonally (Fig. 4). The gradient at pools and transitions is approximately uniform at such HW stages. When LW flow sets in, the water meets least resistance in the deep channels at the concave side of bends and, conversely from before, the detritus is carried principally from one concave shore to another. Consecutive bends are thus separated by a diagonal band of detritus. Backwater occurs above the bar and causes a steep gradient at the crossing, thereby washing particles of sediment over the transition into the following pool (Fig. 5). Some time after the beginning of a LW stage, a state of equilibrium develops and detritus movement ceases almost completely.

In short, the following order of events takes place in a natural water course (Fig. 6 a and b). At HW an excessive deepening of the pools occurs; this development is supplemented by a growth of the bars at the transitions. An increase in the depth of the pools, of course, is of no value to navigation. The growth of bars at crossings does not harm navigation at HW because the water stage rises much faster than the crossing bars. The harmful situation occurs during the following LW period. Thus, at HW there is a betterment in navigation conditions. During this period the gradient of the river becomes uniform and, in spite of an increase in the river bed irregularities, a relative decrease in depth differences occurs, causing a uniform movement of detritus. The more the HW drops, the more variation in slope will occur; a flat gradient develops over the pools, a steep gradient over the shoals. The scouring force and transporting force decrease, but relatively more in pools than over shoals. (The condition is not quite correctly expressed in Fig. 6. The HW line should be about 2 or 3 cm. higher.) The pools become partially filled with gravel at LW, while at the same time the transitions are partially scoured out. In spite of this, the depth at the transition becomes very small compared to that at HW, because

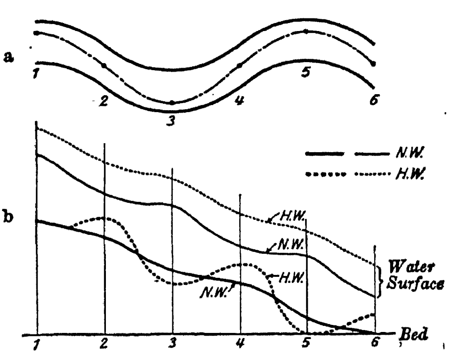


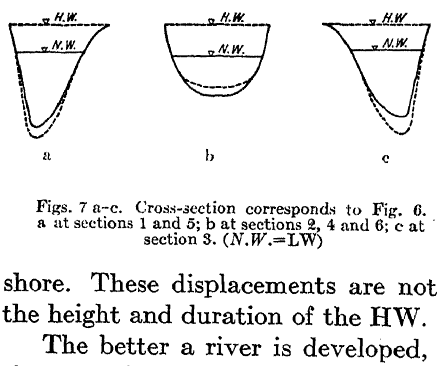
Fig. 6. Thalweg (a) and longitudinal section (b) of a serpentined river course during HW and LW (*N.W.* in drawing)

scour of the transitions is much slower than the rate of drop in stage. Nevertheless much is gained in having the transitions scoured instead of raised during the period of LW. The scouring force thus works to the advantage of navigation.

The mode of occurrence of detritus is important. It is formed by the weathering and wearing down of mountains in the upper river stretches. Large pieces of sediment are carried down the valley, and the further the material is carried into the valley, the smaller it becomes.

However, observations have been made indicating that in the lower course of rivers the sedimentary material is not made up alone of material from the mountains, but that the moving detritus consists mostly of material scoured from the shores of the river's own lower course. If this observation is correct, a question immediately arises regarding where the detritus from the mountains actually remains. It must, in some manner, be taken up by the middle course of the river and, in the course of time, cause a rise of the river bed, the deposited sediment being replaced by material eroded from the shore. Furthermore, this conception does not contradict experience, since for extreme HW of unregulated rivers the flood plains are often covered with sand. Portions of the sediment from the mountains, nevertheless, are eventually ground to dust and, as such, wander into the sea.

Many important observations have been made with regard to the form of river courses. It has been found that wandering banks occur more particularly in weak bends or straight stretches. Wandering banks are those which change their location in the course of time so that the *thalweg*, that is the line upon which a freely floating body is driven downstream, continually changes. In straight stretches the banks move concurrently downstream after every HW, remaining on the same shore. These displacements are not uniform but are dependent upon the height and duration of the HW.



Figs. 7 a–c. Cross-section corresponds to Fig. 6. a at sections 1 and 5; b at sections 2, 4 and 6; c at section 3. (*N.W.*=LW)

The better a river is developed, the more permanent are the locations at which banks repeatedly appear. The stability of the banks, nevertheless, will be disturbed by the following occurrence. The higher the water stage in the river and the greater the resulting velocity of the water, the greater will be its transporting force. During high stages all movable sediment in the river is forced to wander. Ship operators on the Oder have noticed that in order to reach a fixed bed with a guide rod at HW stages they had to penetrate "an almost floating sediment layer" 30 cm. (1 ft.) thick. When the stage lowers, the transporting force is not large enough to move the larger particles. These are first deposited and are followed by smaller grains and finally by the sand. Consequently, river sections are frequently found in which complete sorting of the sediment has taken place. A clever dredge operator who

knows the river in which he is working can usually dredge fine or coarse sand according to requirements. This natural sorting has favorable influence in transitions because the finer material which lies at the top is more readily scoured out by the weak current. Conversely, there is a limit to the depth to which a transition will be scoured, when all grades of sediment are present, because after a certain depth is reached, the sediment is too coarse to be moved by the water. Therefore, at a definite depth the transitions are "armored." If this armor is penetrated by dredging, there is danger of starting banks to wander which were formerly fixed.

Bars occur in practically all sediment-carrying rivers, but they are especially pronounced below the confluence of tributaries. There is usually a break in gradient at this point causing a disturbance in the main river. Moreover, masses of detritus are usually carried to the river, which are greater than the corresponding addition of water. The newly added material must again wander until it reaches regular storage places. Even though improvements bring about a certain degree of fixedness of the sediment bars — reappearance of the same navigation channel after every high water — detritus movement will not be eliminated. With very few exceptions this should not be the intention of regulation, because fixing the channel requires that sediment brought from the upper reaches of the river should be carried further but that it should not be allowed to heap up in particular locations. The condition of bed fixedness has been attained by excellent development of some rivers such as the Weser, while for others, such as the Rhine, in spite of great effort, the sand bars have not been prevented from wandering. The possibility of fixing bars rests particularly upon the type and quantity of detritus, the gradient, the rate of discharge, and the size of the river.

The observation of wandering detritus during HW has been practically impossible up to the present time. Knowledge regarding the mode of transportation, therefore, is very scant and the phenomena urgently require further explanation. In Bavaria and Austria thorough investigations have been made in this connection,[1] because the detritus or bed-sediment problem plays a much more important rôle in the southern European rivers than in the northern flat-land streams of Germany. Nevertheless, there is generally too little consideration given this item. In southern Germany and Austria it has been observed that the HW is not the deciding factor in fixing the detritus movement.

[1] Hartmann: *Untersuchung über das Verhalten der Wasserstände und der Talsohle in der oberen Donau;* Kurzmann: *Beobachtung über Geschiebeführung in der Ache am Einfluss in den Chiemsee usw.*

The effective period of this stage is too short to have as much effect as other water stages for which the movement also occurs. Computations which do not take into account the influence of observed water stages are completely misleading.

The best method proposed for estimating detritus motion requires the following information:

1. The water stage at which detritus begins to move. If no observations are available — they are relatively easily made — an estimate may be made based upon the results of laboratory experiments carried out by Schoklitsch and Schaffernack, two Austrian experimenters.

2. Information regarding the magnitude of detritus movement at several water stages. In case of uncertainty in making estimates it will suffice to use three or four figures which may likewise be based upon theoretical considerations if necessary, provided that one or another of the figures are checked by observation in order to verify the assumptions.

3. The discharge frequency curve upon which the detritus-transportation frequency may be superimposed.

The "bed-forming water stage"— the stage at which the most detritus is transported within the period of one year — as well as the total amount of detritus transported within the chosen period may be read off at the peaks of the frequency curves. An example is given for the Danube at Vienna.[1] The bed-forming water stage may be somewhat changed by construction measures, but the total quantity of material moved must remain unchanged if the bed is to remain fixed and it is not intended to make changes, such as deepening the river bed by dredging operations.

If, despite improvement, the sediment deposits regularly in banks found at the same location after every HW, detritus movement does not cease since material is continually added from above. A condition of bed fixedness has been attained in the Weser. Transportation of bed sediment occurs during HW but the movement has been regulated to the extent that it is not recognizable at LW.

In general, the gradient of a river is in accord with the type of detritus forming its bed. In the upper reaches where the bed sediment is very coarse, the gradient is steep, thereby making the river capable of moving large stones. As the slope decreases the sediment generally becomes finer, there being a definite agreement between the composition of the detritus and the slope. Should breaks in grade occur due to transverse rock reefs, the slope is forced to change, and under such circumstances the general rule is not valid. Such locations are particularly in need of regulation.

[1] R. Reich in the *O. W.*, 1919, p. 482. See also Figs. 23 and 24.

The purpose and measures of river regulation vary according to the type of river or river section. In the case of an unnavigable mountain stream, of course, consideration is given only to protecting land from harmful overflow, swamping, or too great sinking of the ground-water level. Not only is it important to protect adjoining lands, but also to guard the river against the addition of new quantities of detritus. Regular cross-sections are necessary only when a uniform depth of water is desirable; that is, when the river is to be made navigable. If the river is not to be navigable, uniformity in the bed is of less importance, provided there is assurance that the damages arising to neighboring lands are unimportant, and provided the irregularity of one stretch does not cause serious disadvantages to following stretches. In all cases, care should be taken not to handle the river too forcibly, as unforeseen damages usually occur where they are least expected. It is generally useless to carry out the work with great mathematical precision. The statement that one deals with "the art of hydraulic construction" is still valid, although some progress has been made in that definite rules and laws have been recognized. Engels has set up a series of "guiding principles" which clearly give the experiences of recent decades. These, translated from the German, are as follows:

a. Previous to all construction work, the nature of the particular water course must be carefully investigated and the structures made to suit the nature of the course.

b. It is not sufficient to stop certain damages such as shore erosions, and the like, by improvements, but care must be taken to recognize and analyze conditions which have caused these damages. Permanent success of work which deals with overcoming certain actions can be expected only after the causes have been successfully analyzed.

c. The water flowing in the channel should cause the least possible amount of damage at HW and the greatest possible good at LW stage.

d. Forceful encroachment upon existing conditions should be avoided if possible. Where feasible, it is much better to suit the existing conditions. Split channels form an exception, and if at all practicable, they should be removed by damming up the secondary arm or arms.[1]

e. A general increase in detritus movement (scouring force) is necessary only in exceptional cases. Sediment bars which appear during LW — notwithstanding the widespread and disastrous opinions to the contrary — are not an indication of insufficient detritus movement.

f. Dredging, alone, is no method of regulation, because it only retards the action without removing the causes.[2]

[1] Frequently, split channels of a large river must remain undisturbed.

[2] The statement that "the dredge is the dike of the future" has not been verified.

g. In the case of navigable water courses in which improvement by the use of structures is intended (especially for the deepening of the channel) it is wise not to leave the influence entirely to the structures but to assist the action by dredging. If the structures are planned correctly, the dredged depth will persist.

h. Sudden decrease in velocity is harmful because the surplus kinetic energy results in harmful scour to the bed. Therefore, abrupt change in section should be averted by constructional measures.

i. Where very marked scour appears, regardless of whether caused by natural or artificial structures, an attempt should be made to fix the bed and fill the pool.

Thus the aim of regulation includes improving the flow of the water (*i.e.*, equalizing the quantity of water), reducing the influx of detritus, stimulating uniform and smooth movement of detritus, and, for navigable streams, maintaining sufficient water depth. Some few constructional measures are available to assist in this work. These may be used in shaping the section of the bed, in developing the length of the river, especially its gradient, and in fixing the shores. The greatest difficulties in river regulation are due to the great fluctuations in stage. In many rivers the greatest havoc is not due to bad section or gradient development, sharp winding or unregulated detritus movement, but to extraordinarily irregular water discharge.

B. PRINCIPLES OF THE ECONOMIC REGULATION OF RIVER SOURCES

a. Influence of Forest and Agricultural Land upon Run-off

Fluctuations in discharge develop principally in the source districts, the highlands which are subject to the strongest precipitations. To control the discharge in these districts the engineer must have a thorough knowledge of the immediate local requirements. He must have a perspective of the entire problem. Investigations must include studies of the effect of the forest and agricultural land upon the run-off; of means of influencing ground-water conditions; and, particularly, of possible adjustment of flow by the creation of artificial lakes by means of dams in mountainous regions or by conducting water into existing lakes.

In high mountains the flow of water is fairly well equalized because the snow melts slowly in the late spring. Glaciers are similar in effect to regulating dams.

In the German Alps, however, the use of regulating dams is frequently not practicable because of economic and geologic hindrances. Equalization works of this type often cannot be applied above the middle stretches of these rivers.

The large lakes at the glacier border of Germany (Würmsee, Chiemsee, etc.) are available for this purpose and have been proposed in various designs.

In many cases forests exercise an equalizing effect upon the HW wave of rivers. They retard the flow caused by heavy rainfalls, and slow up the rate of melting of snow. Hence, where mountainous regions are deforested, reasonable reforestation is desirable.

Opinions are divided regarding the influence of forests in mountains. In general, however, forests are considered advantageous. The differences of opinion probably rest mostly upon the neglect of observing the types of rivers for which the advantages of forests have been investigated. For example, the flow of a river may be made more unfavorable in the upper regions by the existence of forests. In case snow melting is retarded long enough so that the run-off occurs simultaneously with that of the melting snow from the higher mountains, then an augmented HW stage will result. Moreover, the assumption is that because of lack of forest, the melting snow in the upper course is already passed when melting in the source district begins. Even for this condition the forests will act favorably during the summer by holding back the heavy precipitation in the principal growing season; the SHW will be made milder, and ground-water conditions favorably influenced. The view held by some engineers, that mountain streams may be regulated by forestation alone, holds true only in exceptional cases. Generally, dams are indispensable in the regulation of mountain streams.

b. Natural and Artificial Means of Retarding Flow

(Regulating Dams, Marshes, Glaciers, Natural Lakes, etc.)

The construction of regulating dams is still in its infancy. Few dams are built merely for river-control; usually the production of electrical energy is an indispensable stipulation. Complete regulation of flow, nevertheless, requires dams, even though the gain of electrical energy is of no significance. Unfortunately, dams are so expensive that within the next few decades hardly a single such structure can be built for the sole purpose of regulating flow. Regulating dams not only diminish the influence of water-stage fluctuation but form large sedimentation basins. Sediment may assemble behind the dam in quantities large enough to fill the reservoir to such an extent that the original value of the regulating works is soon lost.

This is one reason why, in the heavily laden, sedimentary rivers of the high Alps, the use of dams is disregarded. In reference to the possibility of regulation by the aid of dams, the author has in mind the German *Mittelgebirge*. In these mountains a number of regulating dams have already been constructed and appear to be very desirable.

By catching large quantities of water at high stages, it is possible to give off equalizing water during the LW periods of rivers. The dam therefore serves two purposes at the same time. Catching the HW

wave hinders floods, thus diminishing the dangers to river valleys; the reserve supply of water, on the other hand, raises the water stage during the summer, and is of value to the entire surrounding country throughout the LW season. Frequently these two possibilities of regulation are not recognized by those interested in particular river projects.

The effect of raising the water level at low stages of the river is greatest in the upper course, and becomes less the nearer the mouth is approached. This condition is readily explainable when one considers that the same amount of water added to the upper course, for example, in a bed of 20 yards width must spread over a 200-yard width in the lower course. In spite of lower velocity in the lower course, the effect is decreased to practically 10 per cent of that attained in the upper course. It is impossible to regulate rivers by simply constructing headwater dams without simultaneous development of the river bed.

A very favorable influence is provided by large natural collecting basins, such as Lake Constance for the Rhine and Lake Geneva for the Rhone. The greatest rate of flow of the Rhine into Lake Constance according to Soldan[1] amounts to 3,545 cu. m. per sec. (125,000 cu. ft. per sec.); the greatest average discharge on both days of observation (June 14 and 15, 1910) amounted to 785 cu. m. per sec. (27,800 cu. ft. per sec.). Thus, the discharge amounted to only 22 per cent of the highest quantity flowing into the lake. Germany possesses a large number of such lakes and groups of lakes. Many of them may be artificially arranged for reservoir service, although temporary HW stages in the lakes may result thereby.

Careful investigations should be made to ascertain whether the use of lakes to retard HW waves will cause harmful effects. Marquardt[2] points out, for example, that if two tributaries carrying HW waves enter a river one after another, the tributary carrying the first wave should not be retarded by means of a dam. If the first wave is retarded, a higher stage will result in the mouth of the main river when the second wave arrives. Such conditions occur in the Danube, where the high waters of the Traun and Enns have practically run off when the HW of the Inn enters.

High marshes, swamps, and glaciers may have similar results, but not in the same measure. The equalizing effect of glaciers is well known. The HW stages in mountain streams depend upon the water supplied by melting glaciers, the quantity being the greatest at the time of highest summer temperature. Moors, of course, take up large quantities of water after long wet periods, and give off the water again during succeeding dry periods. In case HW precipitations strike a saturated moor,

[1] *D. W. W.*, 1923, p. 127.

[2] Marquardt, Erwin, *Die Methoden des Flussbaues*, W. Ernst & Sohn, Berlin, 1922.

however, no additional storing effect of the moor is gained. The result is the same for a regulating dam; the dam can be effective only when it has capacity available for the absorption of HW. Neither a full head-water reservoir nor a full marsh is of value in diminishing a HW wave.

c. Obstruction of Torrential Streams

Although, as has already been said, dams may be effective in holding back detritus, successful obstruction of torrential streams cannot be attained through construction of dams alone.

The torrential stream is formed by individual source streams. A source stream may be divided into three districts (Fig. 8): the upper district, or collecting funnel, in which the slopes are continually being worn down; a middle district, or ravine, in which approximate equilibrium exists; a lower district, the detritus cone, in which the sediment which was loosened is deposited. This lower district usually lies close to the mouth of the source of the torrential stream, in a comparatively flat, main valley, and contains a flat detritus cone formation usually many hundred feet wide. The individual detritus pieces are cemented by sand and clay particles. The cone is usually characterized by great fertility and often is settled by communities. Often the detritus cone is inhabited, which makes a lively wandering of detritus and further sedimentation thereon undesirable. Therefore the aim of obstruction of torrential streams is either to diminish the breaking off of detritus, or to catch the material where it breaks off and where it may be stored without damage. Where the

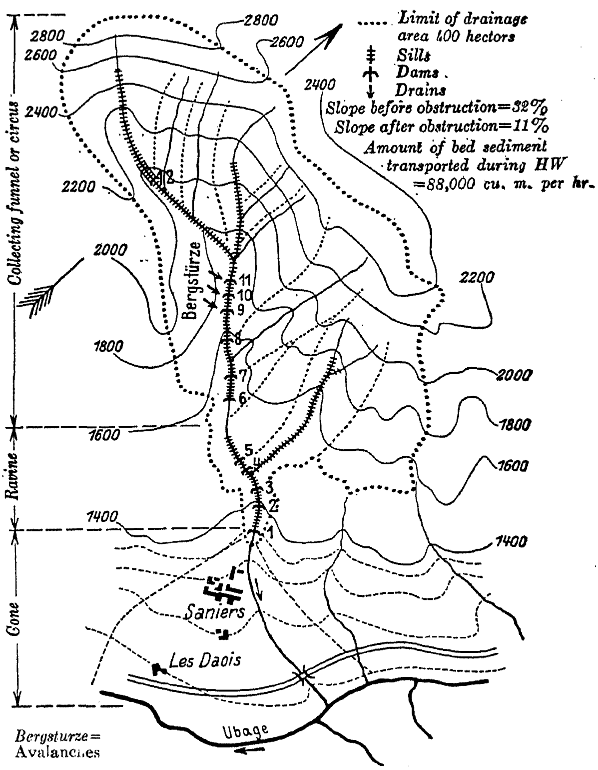


Fig. 8. Torrential streams of Sanier

slopes are covered with forests or other vegetation, wearing away is usually very slow and obstructions are required only in especially steep localities. In localities where there is no plant growth the slopes must be stabilized. They should be transformed into a step form by constructing mat fences backed up with rubble, thus forming cascades which break the force of the water (Fig. 9).

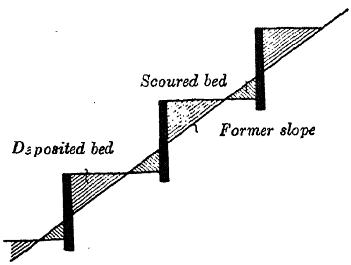


Fig. 9. Cascade for a torrential stream

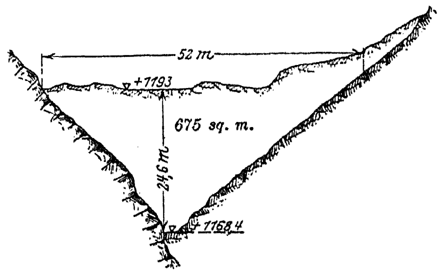


Fig. 10. Cross-section of the Nolla at Thusis showing an erosion of the valley of 25 m.

Serious erosion also occurs in the river bed (Fig. 10). Fundamentally, the same system of correction is applied there as on side slopes. By constructing transverse dams at intervals, the stream is transformed into a water stair. The dams are built of wood in wooded regions, otherwise of stone. In planning a development, investigations should be made to determine the type of construction which is most economical over a long time period, considering the kind of material at hand, type of inhabitants, and distances of transportation. Wood construction, of course, may be made at a very low first cost, but consumes such large sums for maintenance that frequently stone construction is prefer-

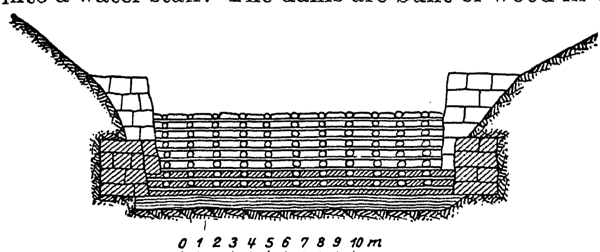


Fig. 11. Elevation

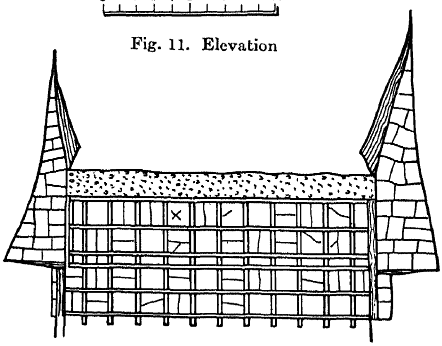


Fig. 12. Plan

Figs. 11 and 12. Obstruction for torrential stream (showing a type of mattress cascade)

able though the cost of installation is higher. This material always plays an important rôle from an economic standpoint, except possibly when communities are seriously threatened; in this case the type of construction giving most rapid relief should be adopted. Figs. 11 to 15 show projects of dams of wood and of stone.

Dams are not in all cases suited to assure catching of the detritus. After heavy rainstorms in valleys and during HW waves after snow-melting periods, so-called detritus avalanches consisting of water and rock roll their way with irresistible force over the sedimentary cone. In especially unfavorable cases they dam up the mountain stream. Serious floods above the barrier result from such detritus streams.

The natural prerequisites for detritus avalanches in the reach of torrential streams are the following:

1. Loose ground, old moraines, old terrain, detritus on the slope, and readily decomposed stone.
2. Steep angle of slope of the mountain sides.
3. Occurrence of HW catastrophes.[1]

The influence mentioned under No. 1 is especially noticeable when a valley separates two districts of different geologic formations, such as Puster valley, where on one slope there is argillaceous mica and on the other, dolomite. Toblacher field is an example of this condition, where in 1856 great sedimentary deposits were carried over the town of Vahlen.

Means to prevent the dangers of detritus flow are similar to those applied by nature. A waterfall corresponding to a steep valley step consumes the transporting force of the torrential stream and causes the formation of a detritus cone. The engineer takes advantage of this simple fundamental principle by artificially changing the slope of the torrential stream into a number of steep steps and very gently sloped, intermediate stretches. The kinetic energy of the water is destroyed as the water falls over the steps, and the flat stretches at the same time serve as a number of artificial detritus cones. The technical possibility of carrying out a development of this type depends upon the geologic formation of the substrata. The forest has also been accredited as

[1] Further examples of the action of detritus streams are as follows: 1471 to 1776, destructions in the French south Alps devasted almost three-fourths of the available agricultural land. From 1836 to 1866 there was a loss to 25,000 settlers in the district of Haute and Basse Alpe. On a smaller scale, 1921, in Sölden in the Oetz valley, Tyrole, most of the houses of the town were damaged by a detritus stream from the Rettenbach. Further, in 1891 a detritus stream in the Eisack valley at Kollmann through the stream Gunderbach (13 sq. km. drainage area) caused destruction of 16 houses and a loss of life amounting to 39 within a few minutes. There was a detritus avalanche of 18 m. height and 500,000 cu. m. content covering approximately 7 hectars. Porphyry blocks of 25 cu. m. content were transported.

having great influence. Holding back water by means of forestation, nevertheless, soon reaches a limit when heavy showers[1] occur. In estimating the value of forests one must differentiate between normal floods, due to regularly reoccurring snow-melting periods and rainstorms, and the out-of-the-ordinary local cloudbursts which occur once in a long period of time. For unusual circumstances, the retention capacity of forests fails to hold back the water.

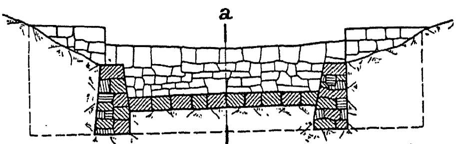


Fig. 15. Section b—b

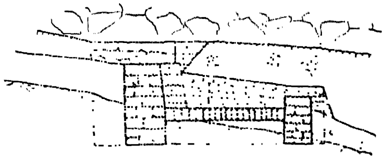


Fig. 14. Section a—a

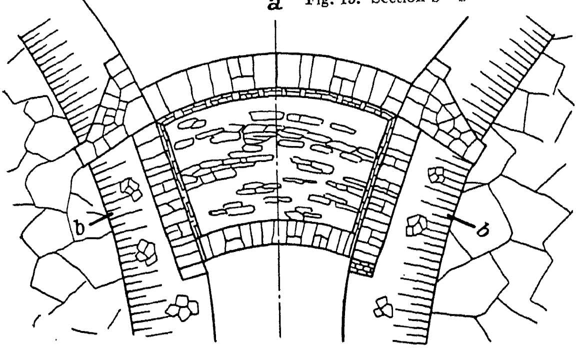


Fig. 13. Plan

Figs. 13–15. Torrential stream obstruction showing a type of rock dam

Dams, reservoir ponds, obstruction of torrential beds, and the stabilization of slopes are necessary for the regulation of HW stages and also for forming protection against unusual occurrences. Forests are only conditional protections. In Germany retention basins and large headwater dams are practicable principally in the Mittelgebirge. The proper use of retention basins in the high Alps is still largely problematic.

Drainage and removal of swamps in meadows and marshes assist in holding back the water. Although the extent of meadows, marshes, and pasture land in the mountains is generally smaller than the area covered by forests, it is still advisable to consider them. In its original condition, a high marsh may be compared to a saturated sponge which simply allows all further added water to flow off. A drained marsh corresponds to a dry sponge which soaks up the water and gradually

[1] This is confirmed by the results of 1882 in Puster valley and by investigations of high flood waters in Silesia of August 1897, both being in districts of thick and excellent forest growths.

allows it to flow away again. Since the especially dangerous moraines of the Alps are characterized by extensive moors, they are drained to utilize the sponge effect.

For practical purposes, the high and low moraines of torrential streams may be differentiated from each other. High moraines are found above the tree limit; low moraines are below the tree limit in the range of thick tree growth. The complete obstruction of a high moraine, if expected to be successful, involves construction work to the very highest points. Because of the high cost incurred, the development is not always practicable. Low moraines are particularly numerous because of the clearing of trees, landslides, and high floods over especially deep detritus heaps at the bottom of slopes. In most cases they can be transformed into satisfactory water channels by pertinent obstruction, by planting the loose slopes with willows, and by reforestation.

Europe has several examples of high moraines. The torrential stream above the town of Oetz, which threatens the fields and dwellings in times of heavy rains, is said to have sent a flow of detritus down the valley as a moraine two hundred years ago. The most dangerous location below the tree line where breaking down occurred was then overgrown with trees, while the upper end reached far into the Oetzer Alps. In 1817 a snowslide tore up the ground and devastated the forest. After this, annual progressive devastation of the valley bottom began. The largest and most dangerous torrent of the Oetzer valley, the Oetzer moraine, likewise has its source within the tree limit in a detritus heap at 2,400 m. (7,870 ft.) elevation. Numerous fields and houses of the town of Oetz have gradually been buried. A masonry dam erected at the exit of the ravine not only became useless, but broke and added more dangers.

Low moraines are ordinarily less common than high moraines. Their activity is evidenced by large detritus hills in practically all of the Alpine valleys. The ruptures in the mighty ground moraines are much more dangerous than the caving of detritus on the slopes. During the spring thaw the material of the ground moraine becomes saturated with water and slowly slides down the valley, with frequent serious danger to towns on the lower reaches of the stream.

In conclusion the following summary is presented:

1. In prehistoric times during and after the melting of giant glaciers, the transportation activity of moraines was one of the preliminary causes for the formation of movable detritus masses, the development of a plant cover, and the later habitation of the mountains.

2. The principal and most permanent devastations in the mountains have been due more to the movement of detritus masses than to

HW. Holding back this material in its original position or in qualified locations in the valleys is the chief problem of regulation. The water-retention capacity of a well-kept forest together with the drainage of meadows and marshes gives satisfactory control of the regular seasonal thawing and rain periods, but is of no practical consequence in times of unusually high run-offs. Among other things, for satisfactory regulation, a thorough knowledge of the masses of sediment encountered and an accurate geological survey of the locality are indispensable.

The foregoing statements are also true for the middle course or ravine into which moraines may enter and in which ruptures generally can occur only by underscour. In this section of the stream, however, the dams must be built larger in view of the greater amount of sedimentary material. The fundamental principle of regulation, namely, the creation of waterfalls, is the same although, since the slope of the valley is flatter than in the upper district, individual steps in the middle course may be made to extend over longer stretches than in the upper course. The stream should be made to flow in one channel over the sedimentary cone instead of in its original state (which usually consists of several arms). The channel should be protected from scour and made as impermeable as possible in order to hinder the water from sinking into the bed. Water which sinks into the bed softens the rubble heaps, loosens the stones from each other, and may make them lighter because of buoyancy. This action tends to set a portion of the cone into motion under the pressure of a moraine coming from above, which would cause serious devastation. Consequently, stream beds are frequently lined with concrete so as to provide complete impermeability. Jointed stone linings have also been used (Figs. 16 and 17).

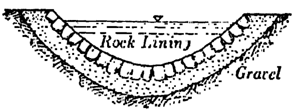


Fig. 16. Rock lining for torrential stream

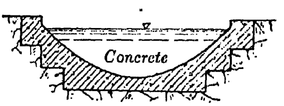


Fig. 17. Concrete lining for torrential stream

C. REGULATION OF UNNAVIGABLE RIVERS

The continued development of navigation on a larger scale has resulted in the elimination of a large number of smaller rivers from the group of navigable rivers[1] and makes it very improbable that other small rivers will come into the group of navigable streams in the future. Most mountain streams are grouped as unnavigable rivers, first, because

[1] The lower Leine River as far as Hanover, the Saale at Wetzlar, etc.

of insufficient flow at MW and LW stages and the large variation in discharge, and furthermore because of the steep slope and the resulting high velocity. European rivers, such as the Leine, the upper Aller, the Lech, and the Isar, most probably could be made navigable by canalization, but they cannot be successfully regulated even for very small boats. All regulation work on these rivers is, therefore, comparatively simple because navigability need not be considered. The improvement of these rivers is now governed by the requirement of fishing, land development, city economics, industry, and power development. In the case of power development, power steps are frequently designed in such a manner that canalization for navigation is a by-product. A plan of this nature has been made for the German Middle Isar project. Power development will, therefore, be discussed in connection with the canaliza tion of rivers. Highland rivers usually have such great variation between HW and LW and such a short period of MW stage that the development of three different beds is not necessary. The discharge of small, unnavigable rivers, which are not highland streams, is usually so small at LW that the development of three beds in the channel likewise is not remunerative. In each of these cases a HW bed and a MW bed should be developed. The lower part of the MW bed may be used for LW without especially fixing the LW channel. A bed development similar to Fig. 18 will satisfy all requirements. Of course, there are exceptions. The Oder section (Fig. 32), for example, has the MW and LW bed stabilized for the purpose of making the stream navigable.

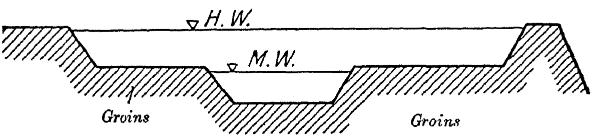


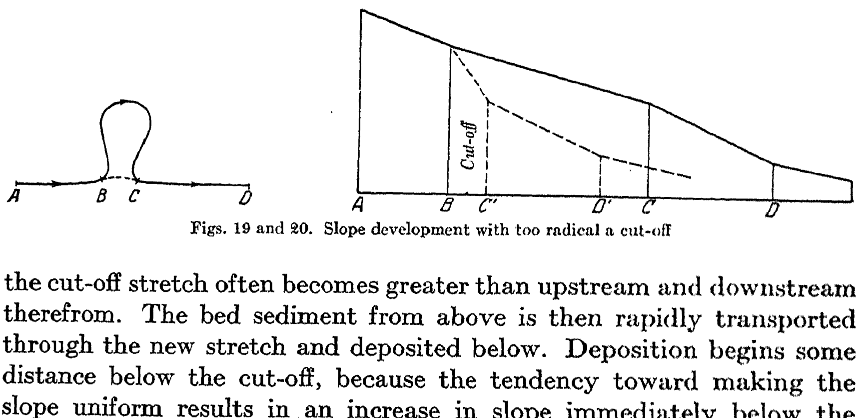
Fig. 18. MW and HW beds

The foremost problems in developing unnavigable rivers consist in providing protection from harmful floods and swamps insofar as regulation deals with caring for the quantity of flow, and then in providing for satisfactory transportation of the detritus so that neither deposition nor scour occurs.

The Isar regulation exemplifies the severe disappointments that may be met with in case of incorrect regulation. The middle-water bed within and below Munich was made too narrow, causing the river to wear down its bed as much as five meters. (At the inlet weir of the middle Isar, the depth of bed erosion amounted to eight meters.) The scouring force of the river was much too great here and, as a result, below Freising the heavy sedimentary load was deposited over large areas. The Isar then worked backwards sharply into its own detritus, beginning at the end of this giant heap. These maladjustments

would not have occurred if the middle-water bed had been made somewhat wider. Scour of this sort, of course, is accompanied by pronounced sinking of the ground water. This was considered advantageous in the city of Munich, but it is generally harmful.

Insufficient detritus movement and the appearance of large gravel beds result in deposits causing the formation of swamps on flat shores. In order to provide greater scouring capacity in unnavigable rivers as well as to guard against floods and swamps, cut-offs may be made. These are preferable and more readily made in such streams than in navigable rivers. Nevertheless, for all such works it should be kept in mind that changes at one location in the river cause changes above and below this location. Increasing the hydraulic slope at one stretch causes variation in slope at least above and sometimes below. The slope becomes greater above. Figs. 19 and 20 show the variations in slope which may occur when too radical a cut-off is made. The fall at



Figs. 19 and 20. Slope development with too radical a cut-off

the cut-off stretch often becomes greater than upstream and downstream therefrom. The bed sediment from above is then rapidly transported through the new stretch and deposited below. Deposition begins some distance below the cut-off, because the tendency toward making the slope uniform results in an increase in slope immediately below the cut-off. Erosion occurs below the cut as well as above, causing a drop of the water level in the vicinity of the cut-off. In case the shores consist of swamp land, lowering of the normal water surface is advantageous; otherwise, it may prove disadvantageous. In this case an attempt should be made to equalize the fall in the old stretches. If it is desired to avoid lowering the normal water stage in the upper stretch, an obstruction causing backwater must be built to retain the original level. For navigable streams a lock must be built in connection with the weir.

Cut-offs in unnavigable rivers are made in the following manner when enough scouring capacity is provided by the stream. First, a ditch is excavated, the bottom of which reaches to the bed of the contemplated cut-off. After completion of the ditch, a temporary cross dam at the upper end is broken through at a time of HW, allowing a

portion of the river water to flow through the ditch. Since the hydraulic slope in the ditch is much greater than that in the curved arm, more water will flow through the ditch than through a corresponding size section of the old bend. Pronounced scour takes place in the ditch,

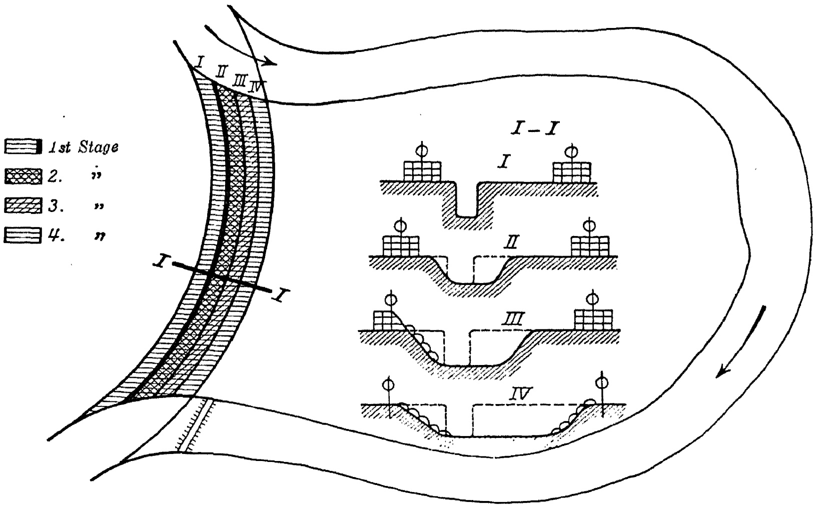


Fig. 21. Cut-off using assisting ditch; cross-sections of the stages of development

and within a short time the river will wash out a new channel while deposition takes place in the old arm where the water flows more slowly. The future shore of the cut-off is piled over with stone in such a manner that the stone will fall into the new bed when sufficient scour has taken place and thus protect the newly formed bed against further erosion (Fig. 21). For very resistant ground the cut may be made to the final shores, which are then stabilized before the river is diverted through the new channel. Cut-offs are readily developed because they have permanent stability. The ditch should preferably be laid nearer the convex shore of the proposed course. Progress of erosion is more rapid toward the concave shore than toward the convex, and the scouring out of the entire new bed, therefore, is more rapid than when the ditch is excavated in the middle.

When the slope (and consequent water velocity) of unnavigable rivers is insufficient, the cut-off must be mechanically excavated over the entire width as explained in the section on "Regulation of Navigable Rivers."

Special experiences in the development of mountain rivers have been

assembled in Bavaria. Specifications have been in force for the regulation of such rivers since 1888. The regulations are as follows:

1. In rivers with high shores the concave shore is to be stabilized.

2. Even in straight stretches, where possible, the normal course is to follow one of the two shores.

3. To obtain a regulated course the shore projections are to be left to the stream to break off and the normal course is to be located as close as practicable to the concave shore.

4. Existing structures and protected stretches of shore are to be given special consideration.

5. If necessary to deviate from the shore of the river because of too great irregularity of the stream, in order to avoid expensive substructures, the normal course should follow islands, immovable banks, and shallow places, insofar as the regulation may be accomplished unhampered.

6. Mountain streams having steep grades are justifiably made completely or approximately straight only when the purpose of the correction is to rid the shore of swamps along extensive stretches.

7. In the remaining cases too much lowering of the water level must be avoided in the interest of agriculture and for assured permanence of shore structures and their cheapest upkeep.

8. The line of correction must be determined in such a manner that existing steep grades are not increased.

9. Hence, a serpentined course of as uniform curvature as possible should be laid out for these rivers.

10. In case the river flows along slopes which contain springs or over which other streams flow, it is recommended not to carry on the correction along this slope, because the desired one-sided stabilization of the shore will probably not be obtained, the resistance against the erosive action of the flowing water being inadequate here.

11. On the other hand, by laying the line of correction in the valley plain, there is a possibility of developing the alluvial shore, which lies between the river and the high embankments, into productive meadows by use of water from the steep slopes.

The HW channel, if possible, should not be contracted if the HW crests already have attacked the foreland. Narrowing the HW channel will cause increased scour. Gathering together the discharge of several river arms into one arm will usually prove advantageous. Frequently the land which is gained by such a measure will be the deciding factor. In cases where the detritus moves too slowly, the necessity of producing better bed-sediment transportation will be the important consideration.

Constant diversion of water from a river has decided influence upon the detritus movement. The diversion may be natural or artificial. Natural diversions often arise because of percolation into the soil (Rhone, Danube) and may lead to large water losses. Fig. 22 shows the quantity of water which flows above Immendingen and the amount

which, because of percolation at Immendingen, is lost and flows to the Ach (tributary to the Rhine) (lower curve).

Large quantities of water are often artificially taken for head-race channels, feeders for navigation canals, etc. The continual diversion of a relatively large quantity of water from a river results in decreased flow immediately below the stretch from which water is drawn. Further downstream, normal conditions may be restored by returning to the river the water which was taken from it, by means of tail-race canals. Diverting water from the river lowers the stage and thereby lessens the transporting capacity. Unless corrective steps are taken, harmful deposition of sediment may be expected immediately downstream from the diversion.

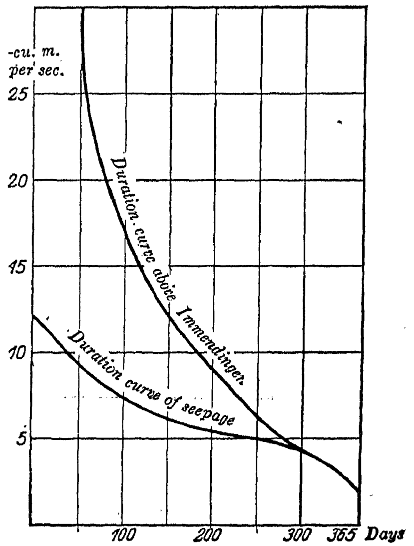


Fig. 22. Seepage from Danube River at Immendingen. The lower curve shows the amount of water which seeps from the river by the time it reaches a point below Immendingen

In the stretch where flow is less than normal the river should be adequately contracted. The requirements are as follows:

1. The new section must be sufficiently large to discharge the quantity of water encountered.
2. Regulated discharge of sediment must result; that is, it must be possible to expect permanence of the section.
3. In case of navigability the desired depth at LW, together with the corresponding necessary width, must be assured.

The following additional requirements concern the best possible aid to navigability (but often they cannot be fulfilled).

4. There should be but little variation in slope of the water surface for various stages.
5. The velocity of flow should not be appreciably greater than in the adjoining stretches.

The river section is designed for one particularly characteristic water stage, inasmuch as one section can hardly satisfy these five stipulations for all water stages. If obtaining regulated movement of detritus is emphasized in designing the correct cross-section, the water stage will be that determined by the level at which maximum detritus movement occurs during a long time interval. This gage height is the so-

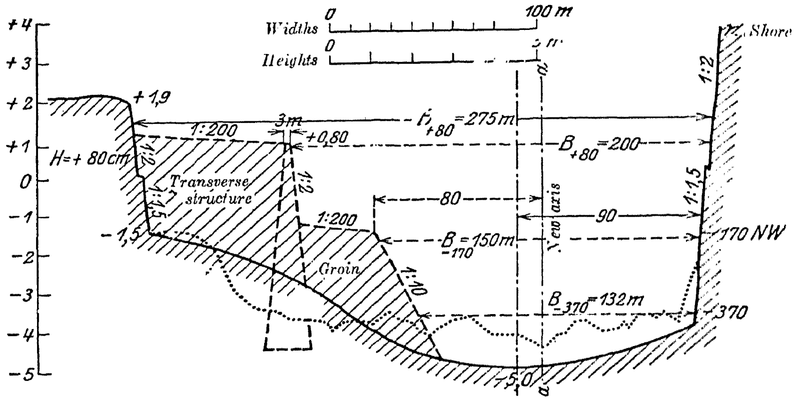


Fig. 23. Constricted normal section of the Vienna cut-off

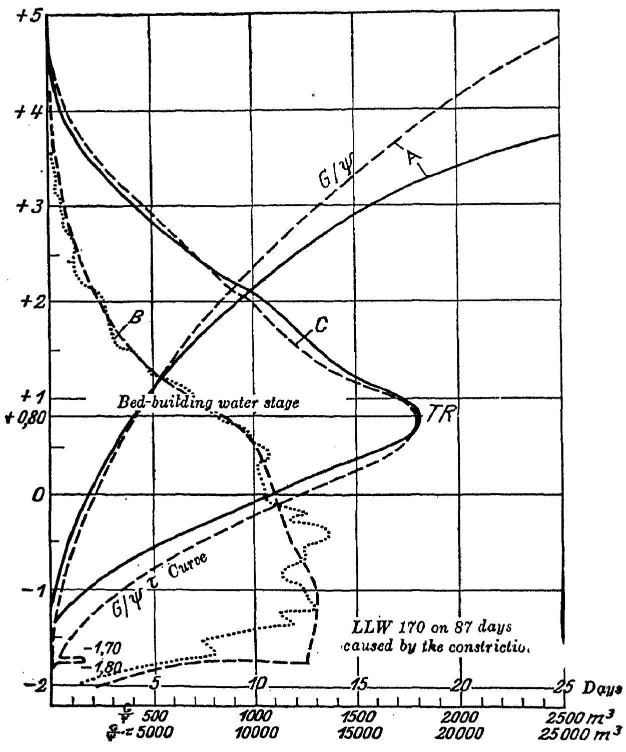


Fig. 24. Determination of the bed-building water stages
A. Relation between detritus transportation in cu. m. per day and water stages. B. Frequency curve of water stages. C. Frequency curve of detritus transportation in cu. m. for the period of investigation
Figs. 23 and 24 refer to the Danube at Vienna. Transportation in the case of continual water diversion . . . . . . before and — — — after transportation

called bed-forming water stage.[1] The numerical methods by which the cross-section is found in such a case, and which is based upon the transporting force theory given by Dubois (used here in Kreuter's form) are found in *Ö.W.* 1919, p. 482. The water discharge and velocity curves are given in Fig. 24. This is taken from an article by R. Reich on the change in the Danube River caused by diverting 400 cu.m. per sec. (14,200 cu. ft. per sec.) for power development. Figs. 23 and 24 are from this article but somewhat simpli-

[1] Schaffernack, *G. O. I. V.*, 1916, Vol. 11/12.

fied. Fig. 23 shows the section of the Danube before (dotted outline of river section) and after the reconstruction. The line $a-a$ is the new axis. In Fig. 24 the frequency curve of the water stages for the old bed is dotted; for the new bed a broken line is used. The curve branches, G/ψ, extending from the left at the bottom to the right at the top, give the variation of detritus transported as 0 to 2,500 cu. m. (0 to 88,000 cu. ft.) with water stages varying from -2.0 m. to 4.7 m. (-6.56 to 15.42 ft.) for the section. (Water stages are indicated at the left in meters.) G denotes the actual amount of transported bed sediment at the section per second for a detritus discharge factor, ψ, which depends upon the type of material. Since these factors are not well known (they are very small), the value G/ψ has been introduced for comparative computations. This gives a correct comparison since ψ does not change with the transformation of the bed. The values of the abscissas in the G/ψ curve are therefore ideal.

The value G/ψ is computed by the detritus-transportation formula (Dubois-Kreuter). Only the method of application will be presented and not the derivation. If the water-stage frequency abscissas are multiplied by the respective ideal detritus-transportation abscissas, the detritus-transportation frequency curve

$$\frac{G}{\psi} \cdot t$$

is obtained, where t is the corresponding time period.

Since for a stage of -1 m. (-3.28 ft.) practically no detritus is moved, the $G/\psi \cdot t$-curve still lies on the left edge; at .80 m. (2.62 ft.) it attains a maximum (horizontal) value and then decreases in spite of the increase in rate of sediment transportation, because of the decrease in water-stage frequency. The crest of the old line (T) and the new one (R) lie practically on top of each other at the stage .80 m. (2.62 ft.). This is the bed-forming water stage. The new bed (Fig. 23) has been designed for this stage (overflow elevation of the MW control works). These imaginary curves are of relative form just as are the actual detritus-transportation-frequency curves. Thus a correct conclusion regarding the position of the bed-forming water stage is obtained, although no conclusion may be drawn regarding the quantity of material moved.

D. REGULATION OF NAVIGABLE RIVERS

Navigable rivers are subject to all of the requirements stipulated for regulation of those not navigable, and in addition the maintenance of a minimum water depth. The latter necessitates uniform detritus

movement. The water of navigable streams, of course, must be discharged without overflowing the banks, just as in other streams. This is always possible if the necessary means are used. Obtaining a minimum water depth, however, is one of the most difficult problems in river hydraulics. A successful solution of this problem requires strict adherence to the first fundamental law of river control; namely, "Give due consideration to the character of the river and do not attempt to force the flowing water to performance which is foreign to the nature of the river." Just as the character of a person must be taken into consideration in order to correct the sharpness which makes him disagreeable, so also the character of a river must be heeded. The latter is expressed especially by its slope; taken strictly, by its energy gradient. The slope of the river gives it the power for water movement; the slope is comparable to the will of man, the water corresponds to the body.

The undeveloped river has large variations in slope, these being the greatest at LW and smallest at HW. Some variations are due to differences in the territory traversed. These differences cannot be changed. They extend over long stretches, and it would probably be wrong to change them because the river has suited itself to fit the lay of the land in the course of hundreds, usually thousands, of years. Most river valleys have been worn down from heights hardly conceivable; for example, traces of the old Swiss Danube are found a hundred meters (328 ft.) above the valleys. The Weser at the Porta Westphalia, the Bode at Thale, the Danube at Passau, the Amazon, etc., all indicate such events in their history. The elevation differences overcome were so great that in individual stretches the river was free to choose its slope. The course developed by the river is more nearly correct than any which man can conceive.

Conditions are different within the individual main stretches. Although the total fall within a loamy or sandy stretch cannot be changed, it is frequently advantageous to vary the gradient locally within the stretch insofar as the form of the river bed may be changed permanently. No attempt should be made to equalize the slope throughout such a stretch, because the original conformation of the river is likely to return.

The river flows in curves connected by more or less straight transitions. Transverse bars or crossings are found between the curves, and deep water in the bends.[1] There is but little fall in the bends, while at the transitions a steeper gradient exists because of backwater due to the crossing bars. The great depth at bends, in spite of smaller fall, occurs because in addition to the longitudinal fall there is pronounced trans-

[1] Fords were as valuable to transportation by land in ancient times as they are harmful to navigation at the present.

verse slope from the concave to the convex shore. The resulting transverse eddies in conjunction with the impact of water against the hollow shore cause erosion and caving of the concave bank. The impact generated by the eddies is the cause of the scoured depression. Such forces do not develop at transitions. In order to develop the necessary depths at transitions, the river maintains a steeper gradient in this locality.

Navigability is determined directly by the crossings and indirectly by the pools insofar as the latter maintain a certain degree of equilibrium with the former. Fig. 25 shows the variation in water stage, water depth, and height of crossing at a definite location in the upper Rhine. The crossing rises with rising stage because, although the depth is greater, the gradient decreases. By the end of a prolonged HW period, May to August, a high sill had developed, the rise being from −0.3 m. (−0.98 ft.) to 1.0 m. (3.28 ft.). From August 10 to October 10, the water stage fell more than 2 m. (6.56 ft.) and the crossing lowered a corresponding amount. There was a rise in stage of 1 m. (3.28 ft.) during October and November and then a drop; the crossing had followed the fluctuation by the end of November. The maximum scour probably does not occur at especially LW but at MW to LW stages, because the transporting force is dependent upon both the depth, t, and slope J. The converse is true in pools; at rising stages the pools deepen and at falling stages they become filled with alluvial material.

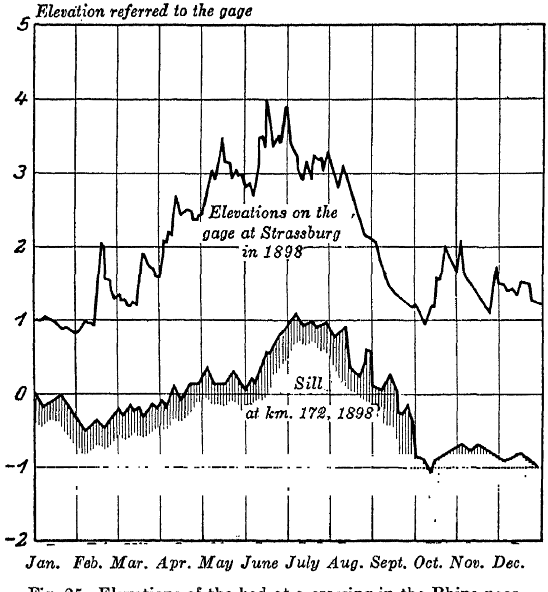


Fig. 25. Elevations of the bed at a crossing in the Rhine near Strassburg in 1898

In treating the slope of a river, cognizance must be taken of the fact that increasing the slope within bends results in further scour in the bends, and decreasing the slope over crossings causes deposition at the crossings. It is not feasible to tamper with the law of the movement of water; local extremes may only be moderated.

The velocity over crossings decreases as the slope decreases and increases at the curves as the slope increases. The slowly flowing water requires a large section, the rapidly flowing water a small one. Accordingly, to make the slope

uniform, the water level would have to be dropped at the bends and raised over the crossings. However, since only a definite total fall is at disposal, no great change can be made in the location of the water surface. Fig. 26 shows this relation very clearly, the figure being somewhat exaggerated. In its original condition the river possessed a water level corresponding to a line passing through points 1, 2, 3, . . . ,7 of the longitudinal section. The slope has been leveled out and the line *a*, *b*, . . ., *f* is obtained, which will be assumed in accordance with the desire for betterment to lie in a relatively higher position over the crossings. The sketch shows that the level would then also lie higher over the bends. Inasmuch as the total fall may not be changed, however, a variation of slope is possible only when the mean slope is retained. Since now the river is to have a diminished slope over the crossings, the necessary depth over them can be obtained only by means of dredging. Even then it is uncertain whether the depth will be maintained by the river; it probably will not, because before dredging the channel the river was not capable of keeping the crossings deep with a greater fall over them.

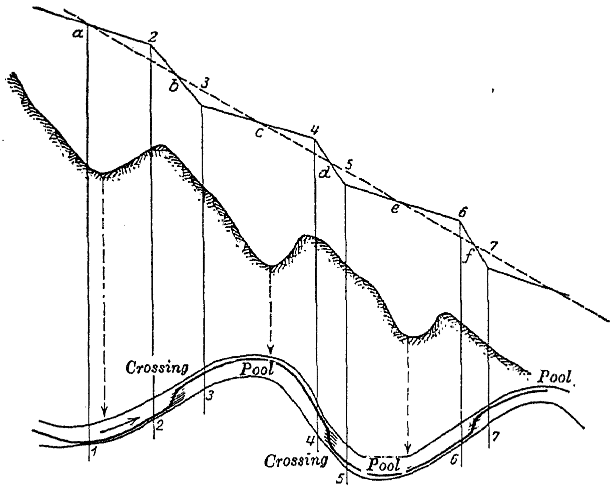


Fig. 26. Crossing and pool formation in a natural river, and the influence upon the water stage

A radical equalization of the slope, therefore, appears to be dangerous. *In order to make the scouring force more nearly uniform, the differences in slope must be locally increased rather than decreased.* In the application of this art of regulation, great care must be exercised to keep the water level from lowering at the crossings. It will do so if the crossing is scoured too much. Sills must be laid at transitions should they become scoured too deeply. If too much lowering of the water surface takes place at the transitions, the level above the transitions will also be lowered, causing the water depth at crossing bars to undergo such a diminution that the condition of the channel becomes worse instead of better. Frequently, it is advantageous to stabilize the bed above and below the transition by sills before deepening is undertaken. This assures rapid scour of the crossing after every HW, because of increase in fall over the crossing, without resulting in harmful lowering of the water level at LW stages.

The first step in the regulation of a navigable river consists in obtaining information regarding the characteristic fall at bends and transitions. It has been observed that in similar types of land a different fall will be developed for each radius of curvature. This relation for the particular river must be taken into account in regulating the river.

The use of many different slopes provides serious difficulties in practice. It is preferable, therefore, to limit operations to a few governing slopes for bends and crossings, and to deviate from these only in especially well-founded cases.

A further question arises as to the method of treating the section and whether the course of the river should be changed. In nature, rivers continually vary their position. These changes in course are the result of mobility of the bed. If the river bed had been invariable in form from the beginning, there would have been no reason for the river to develop its bends and then break through them, and so on. Fixing a new ground plan for the bed, therefore, must be preceded by a choice of the form of bed. The same river in similar sections of country may be given any of a large number of bed forms, none of which can be designated as absolutely the best. Stretches must be chosen which indicate the most uniform and stable conformation. These are stretches which lie in well-formed transitions and well-developed curves in which the slope fits the character of the stretch. Hence, "training-sections" must be sought for bends and also for transitions.

Usually before extended regulation of an entire river stretch is undertaken, test stretches should be built in which permanent forms are erected only after detailed comparative investigations and personal experience. In this way various shortcomings can be avoided and by a cut-and-try procedure, a section can be found which will at least approximately fulfill the requirements, and much unnecessary expense for unsatisfactory arrangements will thereby be saved. Unfortunately, on a number of experimental stretches already built, entirely too little study has been made regarding slope, velocity, and, above all, bed sediment movement. If more care had been exercised, the present status of the art of river control would be much further developed.[1]

The purely theoretical method does not lead to satisfactory success because in theoretical formulas — partly due to lack of observations and partly because, in order to make a numerical evaluation at all possible, so many assumptions and simplifications are adopted that the formulas found thereby have little or practically nothing in common with reality — computations are applicable to only a very narrowly limited range of conditions. The controlling factor is probably not the computed evaluation but the observed one. A method of computation of this

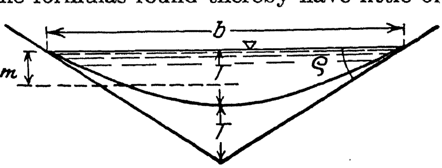


Fig. 27. Teubert's normal cross-section

[1] Compare works by Lagally, Faber, etc.

sort was given by Teubert,[1] formerly chief engineer on the waterways of Brandenburg (Fig. 27). In this method a parabolic cross-section is assumed in which ρ is the natural angle of repose of the shore or of an artificially stabilized bank. When $\cot \rho = m$, we have $b = 6m \cdot t_m$ where $t_m = 2/3T$ or the mean depth of the parabolic section. The values J, Q, and m are assumed to be known, and F, B, and t_m are to be found. By the Chezy velocity formula, $v = c\sqrt{tJ}$; then $F = \dfrac{Q}{c\sqrt{tJ}} = b \cdot t = 6mt^2$, from which $t_m = \sqrt[5]{\left(\dfrac{Q}{m \cdot c}\right)^2 \dfrac{1}{36J}}$.

In this equation c is dependent upon t. Therefore a trial computation must be made for the first assumption of c, giving an approximate value of t. From this computation a closer value of c may be obtained and therefrom a more accurate value of t may be determined, etc., until there is satisfactory agreement. Since c varies very gradually, two trial computations will usually suffice.

The entire method appears too definite. The value of c for a river can be correctly determined only after one has developed judgment regarding the quality of definite sections. Thus, after the work of finding the value of c for the river has been performed, strictly taken, the correct cross-section has been determined. Although very indeterminate, the evaluation of c is simplified by Teubert's method. This method begins with the general coefficients of velocity formulas, such as the Bazin or Hermanek formula. With these coefficients as a basis, theoretically correct sections may be determined, sought out in nature, and then, by varying the section found in nature, an improvement in the c-value can be undertaken. In this way an agreement is reached more readily than if theoretical means of help had not been used. Using the Hermanek formula and considering that for $1.5m < t_m < 6m$ $c = 34\sqrt[4]{t_m}$; it follows that

$$t_m = \left(\frac{Q}{204m\sqrt{J}}\right)^{\frac{4}{11}} \text{ etc.}$$

The value of t_m found by this formula furthermore allows a rough estimate of the probable navigable depth attainable in the river.

Figs. 28 to 31 give examples (to a distorted scale) of the procedure used in choosing a section on the Weser River. The normal section for the location having the steepest gradient was first designed (the inner portion of Fig. 28), and then the sections were determined for flatter slopes, using the same surface width (the intermediate and outer sections). The parabolic depth (depth of the lower parabola) amounts to 30 cm. (11.8 in.) for the stretch above Minden and 45 cm. (17.7 in.) below. All computations are for raised MLW (raised by the Erder dam). According to computations, the MLW is raised .35 m. to .25 m. (1.15 ft. to .82 ft.) from Muenden to Minden; from there to the Aller River, .25 m. to .05 m. (.82 ft. to .16 ft.). As a result of the reservoir, therefore, the water depth will be increased to 1.10 m. (3.61 ft.) at Muenden, 1.40 m. (4.59 ft.) at Minden, and 1.75 m. (5.74 ft.) at Hemelingen. The minimum width of bed is to amount to 30 m. (98.4 ft.).

Fig. 29 shows the proposed sections in mildly curved stretches. In several stretches of the river it is intended to narrow the transitions to a width not less than 30 m. (98.4 ft.) but not to broaden the bends. Figs. 30 and 31 show these cross-sections at the regular and irregular stretches. At the regular

[1] Teubert, *Die Verbesserung der Schiffbarkeit der Ströme durch Regulierung*, Berlin, 1894.

stretches the side slopes are made flatter than computation indicates necessary; at irregular stretches the flat slope is further flattened. Only part of the expected rise of the MLW stage due to the Ader dam has occurred. Probably considerable of the additional water at LW is used in raising the ground water level. Thus, in computing the value of a reservoir for raising LW stage, allowance must be made for the water consumed in raising the ground water level. The latter effect is of no direct benefit to navigation. Additional depth is now being provided by canalization.

The Oder River is being subjected to a system of development similar to that of the Weser. Cross-sections of the Oder are of the shape indicated in Fig. 32, which is supplemented by the accompanying table. The surface width at MW elevation increases from 34 m. (111.5 ft.) at Ratebor to 94 m. (308.4 ft.) at the Silesian border, 150 m. (492.1 ft.) at Warthe, and 188 m. (616.6 ft.) at Achwedt. The depth at LW is 1 m. (3.28 ft.); at MW, 1.62 m. (5.31 ft.). The bottom of the bed is assumed a straight line. The depths are to be increased by water from an artificial reservoir at Otmachau. Success appears doubtful.

Sections may be improved by varying the curvature of bends. The first important success in regulation can be attained by cutting off or reducing the curvature of sharp bends. Even large errors in design, compared to the arbitrariness of nature, do not prevent such measures from effecting a decided betterment in the river. The most difficult work of the engineer arises in refining the roughly regulated river. It is much more difficult

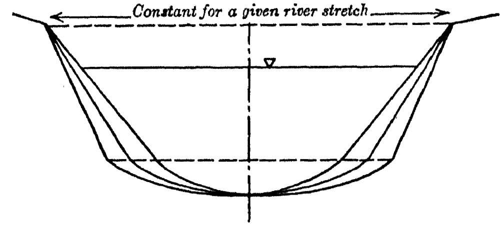


Fig. 28

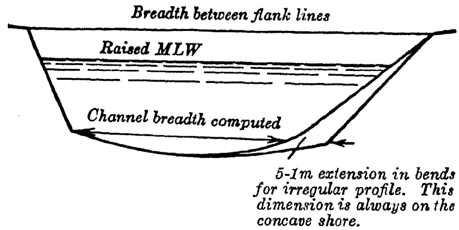


Fig. 29

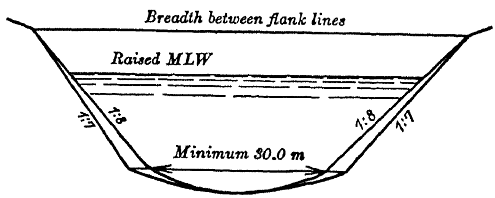


Fig. 30

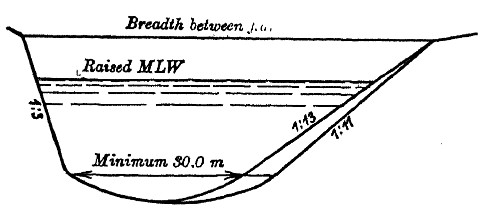


Fig. 31

Figs. 28–31. Normal development cross-section of the Weser River

to bring about improvements in a roughly regulated river which will increase the navigable depth by inches, than it is to increase the navigable depth by feet in an unimproved river.

Table of Widths of the Oder River (Fig. 32) According to Design of 1881

	MW Surface Widths	*Bed Widths at Bottom*
Oderberg to Ratibor	34	
Cosel	45	
Malapane Zg.	50	
Glatzer Neisse	60	
Weistritz	83	53
Katzbach	87	53
Schles. Grube	94	54
Obra	110	65
Bober	120	70
Goerlitzer Neisse	135	80
Frankfurt	150	90
Warthe	150	94
Schwedt	188	132

Since the depth of water at high stage is invariably sufficient and since the slope is most nearly uniform at this stage, the LW bed is most important in the design of the refined regulation works. On the other hand, the proper procedure of construction is started with regulation of the HW bed. The cross-sections having the best form and greatest stability can be most readily found during the LW stage. The quality of the section may be observed particularly by the direction of flow. If the flow at a given section varies only slightly in direction for all water stages, the cross-section is very stable in form. Since for large rivers the LW bed is often strongly serpentined within the MW bed, the requirement that the direction of flow at HW and MW be as nearly congruent as possible is usually considered satisfactory. The scouring force will then act as nearly as practicable in the same direction for all stages. Serious deviations in the direction of scour usually occur when the HW bed has not been correctly developed, particularly when it is too wide. Although a wide HW bed is advantageous in conducting flood flows, it

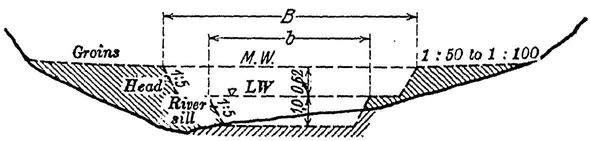


Fig. 32. MW and LW bed of the Oder corresponding to present system of regulation

endangers the development of the MW and LW beds. The HW bed must be sufficiently narrowed by dikes so that flow at the HW stage will follow the direction of the MW flow as nearly as practicable without endangering the adjoining lands or dikes further upstream by backwater. Thus, it is just as important to define the HW shores correctly as to avoid caving banks which increase the movement of sediment.

New permanent forms of the cross-sections for MW and LW are possible only by stabilizing the shore. Bank stabilization at bends requires diminution of erosion. The concave shore usually extends to the bed at a steep slope, the deeper the pool the steeper the slope. The flatter the bank is at the bends, that is, in the vicinity of the navigable channel, the less tendency there is for eddies to arise, and hence the less the danger to navigation. Consequently, the shore should be flattened at the bends. A preliminary requirement for the flattening is reduction of the depth of pools. This causes scour of the cross-section on the convex side, bringing the bed sediment on this side into motion. Below the MW level the convex side usually has a very flat slope which is either a continuation of a sand bank or, in case of badly formed sections, lies to one side of the sand bank, in which case the latter separates two channels. The development of the bed, therefore, requires flattening of the concave shore, reduction of the depth of pool in bends, excavation of the sand bank and thereby steepening of the flat shore on the convex side. Pools may be flattened by building sills across them with fill between the sills. This work usually results indirectly in scouring out the sand banks. Moving the shore, or at least relocating the side slope, and stabilizing it must proceed simultaneously. In general the width of curved stretches should not be diminished, but it may be desirable to widen them. Nevertheless, further narrowing of the section is usually advisable at transitions. The latter measure is much easier than the former, because transitions are much shorter than the corresponding bends. Sometimes one bend flows immediately into another; the transition stretch is then lacking. Here one of the problems of regulation consists of creating a transition stretch.

An apparent contradiction in the foregoing procedure has not been pointed out. The formula

$$t_m = \sqrt[5]{\left(\frac{Q}{mc}\right)^2 \frac{1}{36J}}$$

indicates that the greatest depth will be attained over the crossings when, for a definite stage, J reaches its smallest value. From this, based purely upon mathematical reasoning, it follows that the slope over the crossings should be dimin-

ished as much as possible. Accordingly, it would be attempted to make the fall over the crossings smaller than in the bends. For a definite Q, there would result

$$t_{\max} = \sqrt[5]{\left(\frac{Q}{mc}\right)^2 \frac{1}{36J_{\min}}}.$$

This conclusion would be correct if only uniform flow were under consideration. Actually, however, the flow at crossings is non-uniform. Furthermore, the work of erosion and transportation of sediment after HW and during LW stages is of prime importance. The real problem is to develop a strong current in the transition section, and not a consideration of the advantages of a theoretically possible larger cross-section incurred by reducing the velocity. To offset the decrease in cross-section resulting from rapid flow, sufficient depth must be provided by narrowing the channel at the transition. Even when the section at the transition is completely stabilized by structures, the flow must be more rapid than in bends if detritus movement is encountered. It is correct to diminish the fall at transitions only in rivers where detritus movement is not appreciable. In such cases, however, dangerous crossings are infrequent and the stabilization of transition sections does not provide special difficulty.

The investigations under consideration are explained with the aid of figures showing the effect of good and poor breadth distribution upon the location of sand banks. Fig. 33a shows a river with good breadth development; Fig. 33b, with poor development. Fig. 33a indicates a reduction in breadth at transitions and widening at bends; Fig. 33b shows the contrary. The pools should gradually bend from the curves

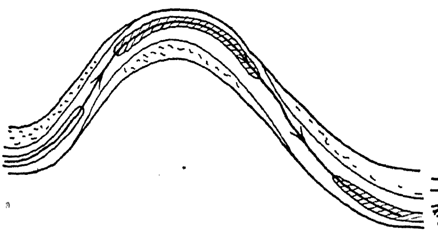


Fig. 33a. Good channel development

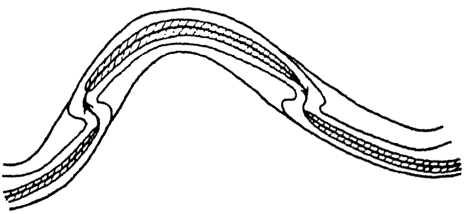
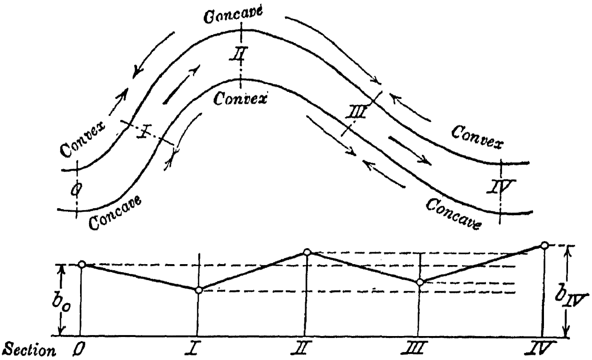


Fig. 33b. Bad channel development

to the center of the river to give good breadth distribution; the sand follows the river course in uniform crescents. A ship may readily navigate from the bend over the transition into the next bend. In a poor development, however, the deep stretches cling to the concave shore over too great a distance and often the ends of successive deep stretches lie immediately across the river from each other, the water flowing in an almost transverse direction over a weir. A ship cannot follow such a course, for it encounters many irregularities in depth. Under the influence of the water flowing transverse to the river over bars, the eddies which arise develop cavities.

Fig. 34 is a good guide for breadth distribution. The following pertinent remark may be added. Every section of a curve is greater than corresponding sections in curves upstream and smaller than corresponding sections in curves downstream from the curve considered. The same holds for transition sections among themselves. Furthermore, every transition section is smaller than sections in the adjacent bends. (This arrangement agrees with early river developments.) The slope will be irregular, flat in the bends and steep in the transitions. This conception is applicable to LW and possibly MW stages in which the river flow is augmented downstream by ground water infiltration. Less significance should be attached to this arrangement for HW. For this stage the development may be made dependent upon local conditions.



Figs. 34a and b. Diagram of breadth development

E. VARIOUS CONSTRUCTION METHODS: CUT-OFFS; RIVER FORKS; MOUTHS OF TRIBUTARY RIVERS; RAPIDS

a. Cut-offs

In the regulation of a river in its natural condition many bends impede navigation, because of their small radius of curvature, and must be cut off. Such cut-offs have been used extensively on the Rhine, Weser, Elbe, Oder, and other rivers. The result of incorrectly placed cut-offs in unnavigable rivers has already been mentioned. Although the effects in these streams are usually not serious, in navigable rivers poorly placed cut-offs may be very destructive. If it is assumed, for simplification, that after a cut-off has been introduced the fall in the lower stretch does not change — a condition which often agrees with reality — then within the cut-off a much steeper slope occurs. The distance upstream which is affected by a cut-off may be determined by computing the draw-down curve. The stretch above the cut-off must be made to suit the new gradient by narrowing the channel. Since sediment which formerly was not in motion is eroded from above the cut-off and deposited below, disturbance in the river may also be

expected below the cut-off. This disturbance may be noticeable for years after the cut-off has been completed. Relief must be obtained by the help of uninterrupted dredging. Excessive expense is involved in all of these measures. Therefore, investigations should be made at the outset to determine whether it is feasible to replace the cut-off by a lock, thereby allowing the river to continue to flow in its old course around the bend.

The retardation to which navigation is subjected at a lock is often more than offset by the extra time required to pass over a long bend. A lock cut-off at bends of several kilometers length sometimes makes water-power development possible. For example, if a river has a fall of 1:2,000 along a curve 8 km. long, a fall of 4 m. (13.1 ft.) is made available by means of a lock cut-off. Of this amount, probably 3.5 to 3.8 m. (11.4 to 12.5 ft.) may be used as effective head.

The method used in making cut-offs in unnavigable rivers, especially mountain streams, in which a ditch is excavated and then scoured to the final size by the natural river flow, is seldom applicable in navigable rivers. The velocity of the water in navigable rivers is usually not sufficient to develop the bed of the cut-off within a short time. Furthermore, the division of discharge between the ditch and the old bend results in a diminution of flow in the bend so that navigation would have to be interrupted during the period of development of the cut-off. Even though flow is sufficiently strong, consideration of navigability makes the ditch method impracticable. For navigable rivers, therefore, a cut-off should be constructed to its entire width and only after complete development should it be opened to the river flow. The construction is usually carried out in the dry to the ground-water level. Dry construction is often used for the final bed by artificially lowering the ground water. Sometimes excavation of a center ditch to a level below the final cut-off bed results in the necessary lowering of the ground water. The ditch itself must be dug either by artificially lowering the ground water or by means of wet excavation. It may be given less fall than the final cut-off and will produce a sinking of the ground-water level especially in the upper half of the cut-off, since it is connected with the river at a lower point where the water stage is at a relatively lower elevation. A flat slope in the ditch is feasible since the ground-water discharge is usually very small. The cut-off remains closed at both ends and the water is pumped over the closing dams into the river. If the cut-offs are short, it is not practicable to lower the ground-water by means of a ditch.

In case work is to proceed in the wet after reaching the ground-water level, the cut-off is opened to the river at the bottom so that a floating

dredge may work toward the upper end. After completion of dredging and laying of the shore protection (also bed protection in the form of sills, when necessary), the upper closing dam is pierced during HW. The tremendous scouring action of the dam masses through the cut-off and into the lower course is usually accompanied by disturbances. However, these disturbances are smaller than they would be if the dam were pierced during LW and allowed to wash away with the gradually rising water stages. The flow through the

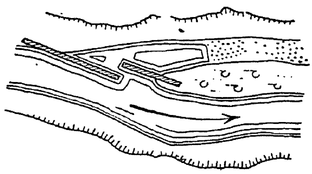


Fig. 35. Closure of a secondary arm with parallel works to obtain gradual sedimentation

cut-off is strengthened (Fig. 35) by constructing a groin in the river arm.

Fig. 36 indicates the procedure used in piercing the separating dam for the large cut-off in the Vistula River at Schiewenhorst. The water made its own course through the separating dam after a small ditch had been excavated.

b. River Forks

On large rivers such as the Rhine, river forks may exist

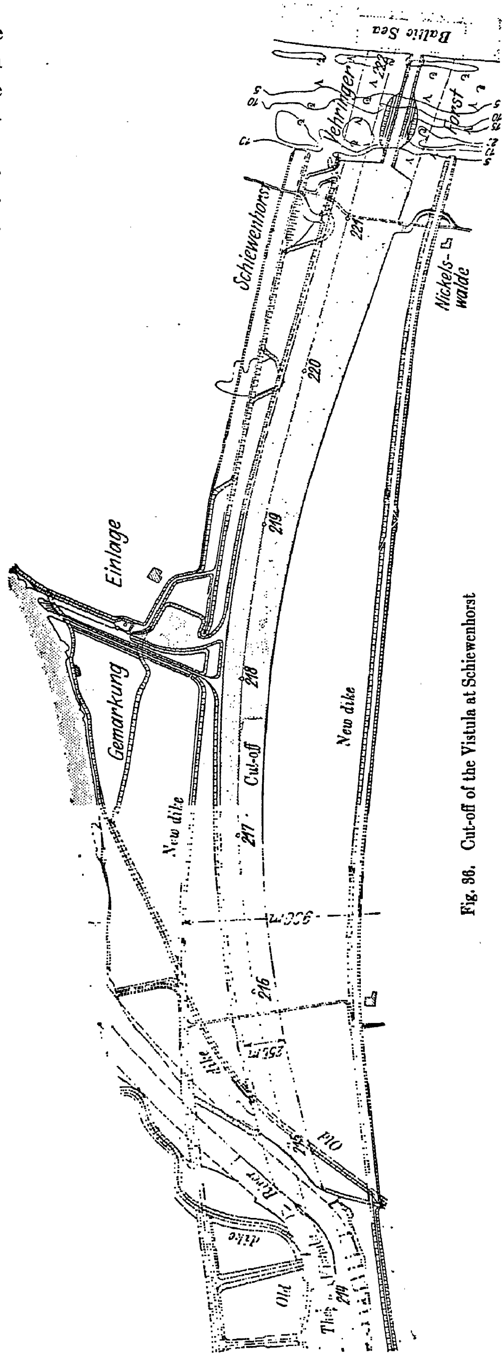


Fig. 36. Cut-off of the Vistula at Schiewenhorst

without hindrance to navigation. However, in smaller water courses, such as the Weser, they may prove very disadvantageous. The treatment of river forks is thus dependent upon the size of the river and the size of the ship.

A numerical investigation follows showing related conditions of a river at a fork and in an undivided stretch. For simplification it is assumed that the relation between the breadth, b, to the average depth, t, remains constant, $\frac{b}{t}=n$. The gradients of the branches are taken as the same as in the main arm. Then for a river arm with a discharge, Q_1, and velocity, $v_1=\mu\sqrt{t_1 \cdot J}$, the discharge $Q_1=F_1 \cdot v_1=b_1t_1 \cdot \mu\sqrt{t_1J}$.

The coefficient μ is dependent upon t. If the Hermanek velocity formula for $1.5\ m<t<6m$ is used, which includes most of the average cases, $v=34\sqrt[4]{t}\sqrt{tJ}$. Therefore $Q_1=34\sqrt[4]{t_1}b_1t_1\sqrt{t_1J}=34\sqrt[4]{t_1}\cdot nt_1\cdot t_1\sqrt{t_1J}=34n\sqrt[4]{t_1^{11}}\sqrt{J}$, from which $t_1=Q_1^{0.364}\cdot\frac{1}{(34_n\sqrt{J})^{0.364}}$. Similar relations may be derived using other velocity formulas.

In reality the fall is usually less in forked stretches than in the main channel. However, it is possible to adjust the fall in the forked arms and in the stretches above and below the fork by regulation. For the foregoing investigation a uniform J is assumed for all stretches, thereby obtaining relations between only two variables since all other members of the equation become constant. Written in a somewhat different form the equation then reads:

$$t_1=cQ_1^{0.364}$$

in which

$$c=\left(\frac{1}{34n\sqrt{J}}\right)^{0.364}.$$

This equation is true for all river stretches having the above constants, regardless of the magnitude of Q. The magnitude of t may now be computed for the principal stretch and then for every arbitrary division of Q. For $Q_1=.5\ Q$, when t represents the average depth in the main stretch,

$$t_1=c(.5\ Q)^{0.364}=.5^{0.364}c\cdot Q^{0.364}=.5^{0.364}t,$$
$$t_1=.777\ t\backsim .8\ t.$$

The foregoing analysis indicates that the disadvantage of forks is that the water depth may be only .777 or about .8 of the depth in the single course, assuming about half the flow goes down each arm.

If the discharge is insufficient at LW, as is the case in almost all rivers, it is desirable to eliminate the forks in the interest of navigation. An additional advantage attained by so doing is that only one river stretch need be maintained, instead of two. A disadvantage is that, in addition to the space taken by the forked arm, the developed arm must

be widened to that of the main channel. The following procedure may be used in determining the width. In accordance with the preceding analysis,

$$\frac{b}{t} = n = \frac{b_1}{t_1}$$

and therefrom,

$$b = b_1 \cdot \frac{t}{t_1} = \frac{1}{.777} b_1 = 1.3b_1.$$

Thus a width of .3 of one arm must be added to the developed arm. The length of time required for the secondary arm to become filled with sediment to the extent that it may again be usable for agriculture is uncertain. The masses of material excavated in developing the main arm suffice only in part to fill the other. If the arms are unequal in size, usually the larger arm should be developed. Where sufficient sedimentation is possible, there is some advantage to agriculture in that a strip of land $2b_1 - 1.3b_1 = 0.7b_1$ wide is gained.

In the case of very large rivers, the relation b/t may be varied. One arm may be developed to serve navigation at LW and MW, while the other may be maintained to discharge a large portion of the HW flow. This has been done at Magdeburg where the by-pass is cut off by the Pretziener weir and is opened only at high-water stages.

In case forks cannot be removed, separating works are usually built at the upstream and downstream ends. Above the fork their purpose is to regulate the amount of flow through each of the arms; below, they serve the purpose of causing the two streams, which tend to approach each other obliquely, to flow parallel so that large eddies and serious sedimentation will not occur at the junction of the two streams. The separating works may reach only to the point at which the width of both arms is equal to that of the undivided course; that is, at the theoretical point of separation. The separation groins must be feather-edged to the ground from a wide section at the island between the arms.

If a fork is to be removed, a closing dam is built at the lower end so as to induce as much natural sedimentation as possible. The separating dam at first is built to such a height that only a small quantity of the MW flow passes the arm, causing the main flow to pass the navigable arm. At higher stages a more nearly equal division of the flow and consequently of the sediment will take place. The separated arm will gradually become filled. Depending upon the rate of sedimentation, the separating dam should be built higher from time to time. If the embankment is raised by sections, it will be strongly overtopped by HW. Therefore, the dam should not be laid upon a natural sill, but at a deeper

point upstream of such a sill. It should be protected at the foot by wattlework. Its location must be so chosen that both ends lie against fixed shores of sufficient height to prevent water from flowing around the ends, thereby underscouring the dam. If such a location is not available, the ends must be particularly well protected. Raising the crest of the closing dam a certain amount periodically, rather than building it to its full height immediately, is favorable to sedimentation in the arm; furthermore the dam need not be built to the width which would be required for one built to its final height in the beginning. Progressive sedimentation takes place on the upper side, thereby bringing about a natural strengthening of the structure.

Where the closing dam is to be built to the final height at the outset, a location as high as possible should be chosen. The site should be so situated in this case that enough dredged material can be obtained to fill the obstructed arm.

The closure of superfluous arms brings about permanent betterment to navigation. However, during the progress of the work a temporary maladjustment may arise, since immediately on closure of a branch the navigable arm must put into motion sand which formerly was stable. For large developments of this type, therefore, it is necessary to have a sufficiently large dredge park for simultaneous deepening and widening of the navigable arm. Work on the new HW bed may be begun long before other operations, because this work does not substantially influence navigability. Construction of separating dams becomes more expensive for immediate complete construction. The complete layout is cheapened by the advanced construction of the new HW bed. The upper diversion of an arm to be filled must be partly closed by a separating groin which should have its crest high enough to cause sufficient flow for navigation to be diverted into the navigable arm at LW and MW stages (Fig. 37).

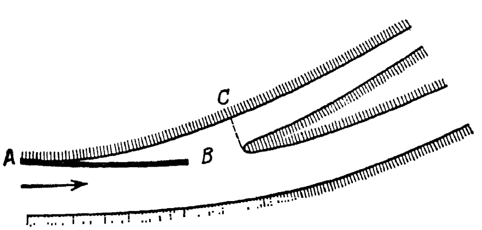


Fig. 37. Separation groin in the arm where deposition is to take place

c. Mouths of Tributary Rivers

The mouths of rivers of the first order (those entering directly into the sea) are treated in a separate section, because for these the influence of the sea is of major importance.

In contradistinction to mouths of rivers of the first order, mouths of tributaries are all on the main stream.

Tributaries must be made to enter into rivers of the first order in such a manner that a large increase in cross-section will be avoided.

From this viewpoint, the junction of two tributaries into a river near the same point will usually be harmful and requires a change in location of one of the two mouths. A displacement upstream is possible when the tributary, as is usually the case, possesses a steeper gradient than the main river. Tangential junctions are considered bad because the resulting section is too wide for a much longer stretch than when the tributary enters the main stream at an oblique angle. Mouths should be so situated that the main stream and the tributary form an angle of about 20° to 25° with each other. Invariably sand banks form below the point of confluence of one stream with another, because the transition is too large at the joining section of the two streams. (This is also the case at the lower juncture of river forks.) The waters of the main stream and tributary remain separate for a long distance below the point of confluence, and since there is no separating bank between the two streams, each water course undergoes a widening of channel in effect. The river regulates this condition itself in that sediment is deposited below the juncture. In wide rivers the deposited sand banks may not be harmful; furthermore, they are in part scoured out during HW stages. Repeated dredging is necessary in the case of small rivers. Dredging is unavoidable because the formation of bars below the tributary mouth is inherent in the nature of the river.

d. Rapids

Rapids occur in localities where the bed of the river is so resistant that erosion takes place substantially slower than further downstream. These places are usually pronounced rock stretches which have for ages been the greatest hindrance to navigation. The Binger pocket (Binger Loch just downstream from Bingen) in the Rhine, the Iron Gate (Eiserne Tor) and the Long Whirlpool (Lange Struden) in the Danube, the Domfelsen in the Elbe at Magdeburg, the Latferder crags in the Weser, and similar stretches contain such rapids. Figs. 38 and 39 show a plan and longitudinal profile of the Latferder crags between kilometer 123.5 and 123.75 in the Weser. In this rocky stretch the course is navigable for a single line of traffic beginning at MW stages. It has the shallowest water when there is little or no gravel on the rock crags. Two-line navigation can be provided only by canalization. The slope differences between these river stretches and normal stretches are usually so large that blasting out the rock would seriously endanger the river further upstream. Here bed erosion would come into operation to the extent that rock formations would come to the surface, which formerly were not a hindrance to navigation. In large rivers it is often possible to blast out a canal so that the water at both ends of the rock bank will

remain at the same gradient as before. This was done on the Rhine at Binger Loch. The ships are then towed on long hawsers in such a manner that only the stream vessel or one of the boats is in the rapids at a time.

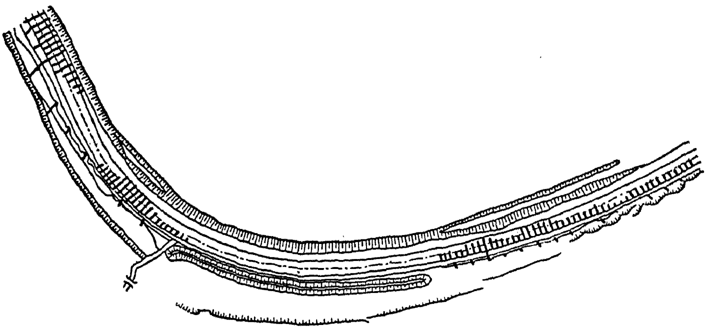


Fig. 38

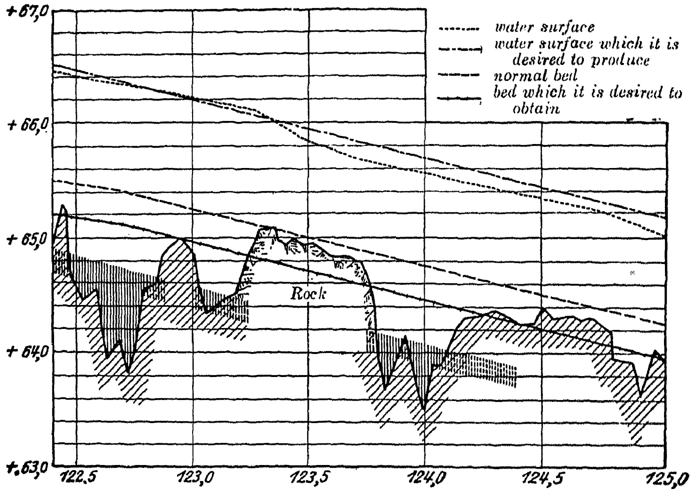


Fig. 39

Figs. 38 and 39. Latferder crags in the Weser River. Plan and profile

The strong current of the rapids may be passed with a towboat of sufficient power. A better solution is obtained by the construction of a side canal with locks so that the flow relations remain unchanged. The construction of a lock with a weir transversely through the river, so that the water level over the rapids will be raised, may also be worthy of consideration. Power development, in conjunction with such an arrangement, often makes the plan economical.

F. MATERIALS AND TYPES OF STRUCTURES USED IN RIVER REGULATION

a. Materials

Rock material for regulation works may be in the form of sand, gravel, or large rubble. Sand may not be too fine, and drift sand is entirely unsatisfactory. In addition to sand, clay may be used if there is certainty that dissolution will not occur while the material is being placed. Wet clay in streaming water can be transformed into a thick flowing liquid in a few hours. Sharp stones are better than rounded ones; crushed rock is therefore better than gravel. Suitable stone can be obtained practically everywhere on the upper and middle courses of rivers, since the latter cut their way through mountainous regions at numerous localities. The type of stone, whether sandstone, granite, limestone, etc., is not important, if the material remains under water continuously. Stone with high specific gravity is preferable to lighter material. The minimum size of stones is best determined by experiment; on an average a minimum of from 5 to 8 cm. (1.9 to 3.2 in.) diameter is used.

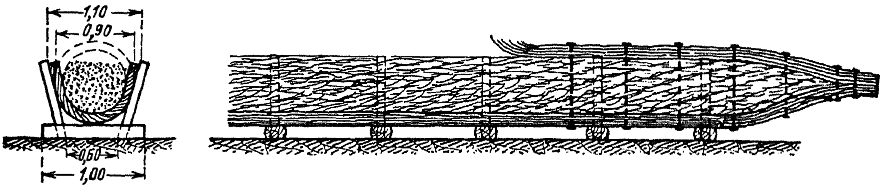
In the vicinity of the lower course and mouth of a river, rock is expensive because of the long transport distances and is usually used only to weigh down brush-work. Brush-work structures are built of fascines, the bundles of which should be about 30 cm. (11.8 in.) thick at the stem end. The fascines should be at least 2 m. (6.56 ft.) long, preferably 2½ m. (8.20 ft.); they are frequently used in lengths of 4 m. (13.1 ft.) and greater. The stem end of individual branches should not be more than 5 cm. (1.9 in.) in diameter in order that the fascines may have sufficient flexibility. Practically all tree and bush types can be used; willow, elm, and ash are best qualified. Thorned shrubs should be avoided in consideration of the danger of injuring the worker's hands. Willow wood has the property that its roots sprout and the twigs continue to grow if cut during the growing season and soon used. This so-called live wood is to be preferred to dead wood. Dead wood must be cut outside of the growing season so that it is sapless. Whether or not deciduous branches have been cut at the right time can be observed by noting whether or not the twigs are leafless and without buds. Needle woods are used with the needles. The fascines are pegged in place with pointed stakes, which are usually about 8 cm. (3.2 in.) in diameter at the top and they can be made of branches or split wood. The first have the possibility of being live wood; split wood, on the other hand, works into the construction more readily. Wooden pegs, usually of oak, are pierced through their upper end. The fascines are made into rolls

12 to 18 cm. (4.7 to 7.1 in.) in diameter. At present the rolls are best bound with double-turned, tempered, and galvanized iron wire of 1.2 mm. (.043 in.) thickness.

b. Construction with Fascines

Fascines are either built in place as packs or mattresses in groins, or they are formed into rolls, fascine bundles, or wattlework on shore. They are then sunk in the river at the location requiring protection.

Abattis and retards are also constructed of fascines. Both consist of fascines laid in layers 10 to 15 cm. (3.9 to 5.9 in.) thick. The layers are fixed by rolls or wire ribs fastened to pegs. The pegs, having a length of about .7 m. (2.30 ft.), are driven at a quadrangular distance from each other of between .6 and .8 m. (1.96 and 2.62 ft.). Abattis should preferably be of willow timber so they will remain green and grow in place. Abattis lie transverse to the stream while retards lie parallel to the stream.



Figs. 40 a and b. Roll mattress under construction showing cross-section and mattress bench

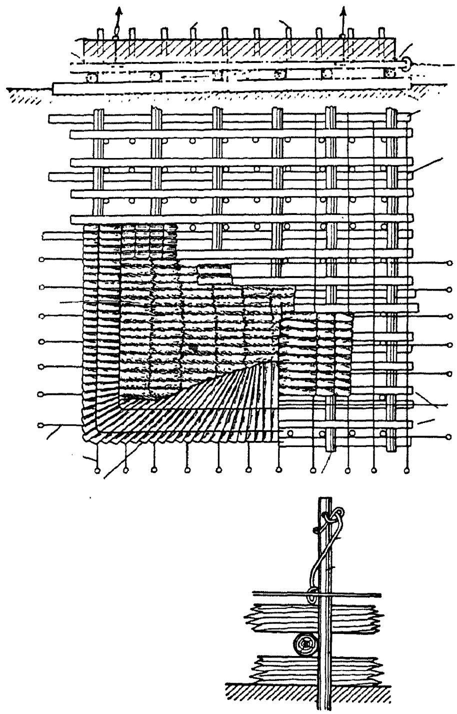
Roll mattresses are made up on special benches. Fascines (Figs. 40 a and b) are spread out on the bench and laid up on the sides so that a long trough is formed with them. This is filled with gravel or small stones and then covered with fascines. The whole is tied together with wire so that a cigar-shaped body is formed. Roll mattresses are between .6 and 1 m. (1.97 and 3.28 ft.) thick and from 4 to 6 m. (13.1 and 19.7 ft.) long. They can be developed into continuous rolls. This is done by progressively tying together the roll on the shore and lowering the finished portion into the water as construction progresses. Another procedure is to construct several rolls one after the other and roll them into the water successively.

c. Wattlework

Wattlework is used where large areas are to be covered rapidly. For rivers, the construction proceeds at the shore on inclined tracks; at the sea, it is done on flats which are dry at LW.

In constructing a piece of wattlework (Figs. 41, 42, and 43), round timbers are first laid on an incline. Planks which are to carry the wattlework are laid upon the timbers. Wire ribs are then laid [at 1 m. (3.28 ft.) intervals], forming a quadrangular system. At each intersection of the transverse wires a "wick

stake" is driven into the ground and the wick or cord is pulled to the top. The wick, a double rope about the thickness of a finger, is thrown over the intersection of the wires. The wattlework is then packed in such a manner that only stems are seen from the outside. The thickness of the mat amounts to .6 to 1.5 m. (1.97 to 4.92 ft.). After completion of the packing, a wire layer similar to that on the bottom is laid on top of the wattlework, and the wires are pulled together by the wicks at the points of intersection. During this operation the workers stamp the fascines together as tightly as practicable. The top and bottom wire beads are tied together at the ends; cross timbers are built into the mat to provide a firm grasp to which hawsers may be tied for pulling the mattress to the desired location, and for holding it in place while being sunk. Woven partitions are built on top of the wattlework, forming coffers to hold material for sinking the mat. The wattlework is rolled off, floated to its desired location, and sunk by loading with stones thrown from anchored boats. It must be well anchored during the process of sinking.



Figs. 41 to 43. Wattlework construction
Fig. 41. Cross-section
Fig. 42. Plan of the grid and wattlework
Fig. 43. Wick stake

Wattlework can be constructed to an unlimited length and in widths up to 100 m. (328 ft.). When constructed in one continuous piece lengthwise, it must be built upon rafts. The raft is moved a distance forward from time to time allowing the completed part of the mattress to be sunk.

d. Dip-Layers

Dip-layers form a part of groins which must be built in deep water. The method of constructing these groins will be treated in the next section. The dip-layers are somewhat similar to wattlework in that they are constructed in a floating condition and are weighted down by gravel

or broken stone. They are not floated to their final location but are built just above the location at which they are to be placed, the land end being begun in place on the finished portion of the groin. In accord-

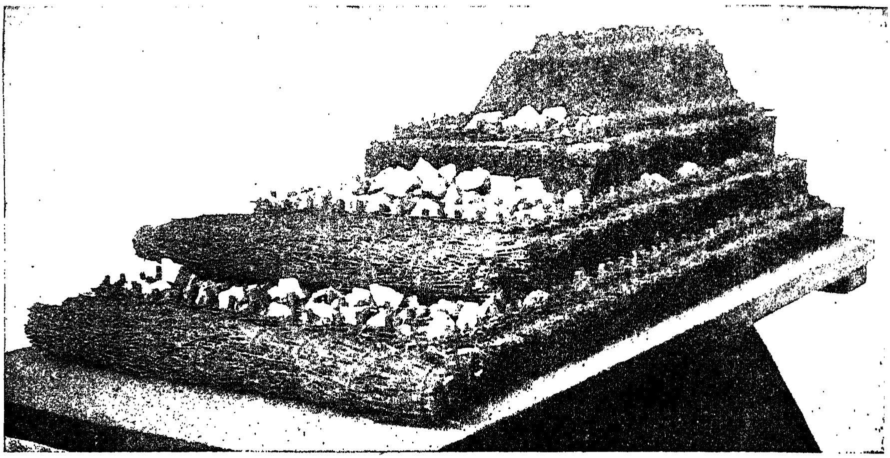


Fig. 44. Model of a piece of wattlework

ance with the method of construction, dip-layers might also be called "trap-layers," because one after the other traps downward under the weight of the superimposed load. The size of dip-layers is limited by the floating capacity of the layer. Wire beads cannot be used at the bottom; the layer is held together by wire beads or fascine rolls at the top.

e. Dip-Trees, etc.

At locations where permanent structures are not built, but where it is desired to induce sedimentation, dip-trees and the Wolf pendulous method are used. Dip-trees are leafed trees anchored to large concrete blocks (Fig. 45). They act as a luxuriant river growth would in retarding the velocity of flow. As a result there is a decrease in the transporting force and sediment is deposited. The same principle applies to the Wolf pendulous method in which layers of fascines are fas-

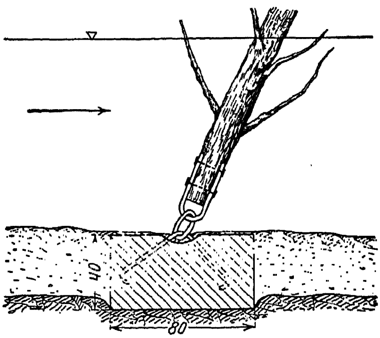


Fig. 45. Dip-trees showing method of fixing tree to concrete block

tened to pile scaffolding in such a manner that they are held suspended in the water. The method of construction of these layers offers nothing new with reference to the used material; fascines of every type may be used. Wire mesh supported transverse to the river on piles (wire groins) is also based upon the principle that retardation of flow causes sedimentation.

G. TYPES AND METHODS OF CONSTRUCTION OF REGULATING WORKS

a. General

Regulating works may be divided into three groups:

1. Fixed restriction works such as groins, parallel works, etc.
2. Movable restriction works such as pendulous structures, floating groins (rafts to cause backing up of water), dip-trees, and the like.
3. Finishing work and channel lining, such as shore protection and bed lining.

The first type, the fixed restriction works, consists of parallel works, sills, groins, and closing dams. They all have several things in common. The construction for all of them must take place on a substratum which is changeable or which may suffer changes directly due to the control works. Furthermore, all of them usually have to be built into the water. Therefore, the structures must be built of fill or sinkable materials, of rock fill or of fascines. In a certain sense, the material to be used in these structures may be said to be dependent upon the wealth of the country. Fascine structures are often built where sufficient means for the construction of stone structures is lacking. Wherever practicable, however, all fixed restriction works should be made of rock fill, while fascines should be limited to pendulous structures. Construction work at river mouths frequently forms an exception to this rule.

Both types can be of rock fill or fascine construction. Rock fill is permanent; fascine structures, temporary. This is especially true for rivers which carry large quantities of detritus (mountain streams), since the heavy sediment gnaws the fascine structures. Bed lining of fascines is applied in the form of wattlework mattresses and rolls.

The second type, movable restriction works, consists of dip-trees and pendulous structures. Both are placed only temporarily and can often be re-used after their purpose has been fulfilled.

The third type, finishing works, consists of shore covering and bed lining. They serve to protect the shore and bed from the attack of flowing water, and form the necessary supplement to restriction works. New deposits which have been created by successful restriction works

must be protected even more than the old shore because the new shore is usually made up of less resistant material.

b. Groins

1. Purpose and Action

Groins (frequently termed "dikes" or "wing-dams" in the American vernacular) and dams of rock fill or fascines, are constructed transversely from the shore into the river to narrow the channel. Success is attained by groins only when the space between them becomes filled with sediment, so that the shore is moved as far forward as the vicinity of the groin head. The action of new groins is similar to backwater effect which would be caused by a series of bridge piers placed one after the other. Between the groin heads, lying on opposite sides of the river, the water flows more rapidly and spreads again in the field between adjacent groins. It has been found that in very rapidly flowing rivers, particularly mountain streams, regulation by groins is not successful. According to data by Kreuter,[1] detritus banks have formed even in the center of the stream between successive groins. Hence, the purpose of regulation is never reached immediately after the construction of the groins, but only after the fields between adjacent groins are filled with sediment to the extent that a new shore is formed extending from groin head to groin head. Good sedimentation can be accomplished with groins only when the distance from the groin head to the original shore is large; very short groins do not offer sufficient protection for deposits. Consequently, groins should be built principally where they can be of sufficient length. These locations are dependent upon the position of the new flanks[2] and lie on the convex shore. The space between successive groins is chosen as either twice the new river width between flanks, in which case groins are built halfway between successive groins after partial filling has taken place; or the groins are immediately located at intervals equal to the new river width. The distance depends upon the type of river; if rapid sedimentation can be counted on, the first procedure would be chosen. The construction cost of intermediate groins will then be less because they can be placed in

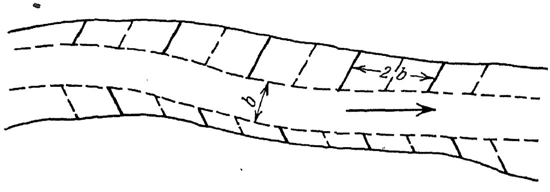


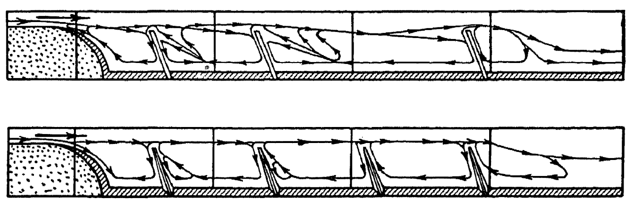
Fig. 46. Intermediate groins

[1] *Handbuch der Ingenieurwissenschaften*, Vol. 3, p. 6.

[2] The flank is the new side limiting line of the river for the chosen water course, for example, at MW or LW.

shallower water. The intermediate groin is needed to increase the rate of sedimentation and to stabilize the new shore (Fig. 46). Groins may be inclined upstream, built normal to the direction of flow, or inclined downstream, and accordingly are called upstream, normal, and downstream groins. In general, upstream groins have proven themselves as the correct system. Their action upon the river bed is milder than that of downstream groins; they allow the water to overflow the groins in a direction toward the center of the river, thereby diverting it from the shore. The attack on the groin field and the shore, therefore, is less for upstream groins than for downstream groins. On the other hand the danger to the groin heads is greater for upstream groins because pools at the heads become much deeper than for the second type. Nevertheless, since the problem of the engineer is to create structures which will provide the best possible result rather than the least amount of difficulty in construction, the danger to the structure is of secondary importance.

In spite of the fact that upstream groins are held as the best, downstream groins should be considered in exceptional cases; for example, in dealing with a river having a resistant bed and shore, the downstream groins will prove to be better, because a stronger attack on the bed is necessary and increased attack on the banks is not dangerous. In such a case it would be wrong to construct groins schematically according to the customary system.



Figs. 47 and 48. Direction of flow lines in fields between groins with groins spaced at various distances according to experiment by Engels

The discharge of water from groin head to groin head does not take place in such a manner that the flow spreads in the groin field up to the middle of the field and then back to the next groin head. On the contrary, the flow spreads to the side below the groin head, and after flowing through the groin field it impinges on the lower groin and is turned toward the shore thereby forming a ring flow within the groin field (Figs. 47 and 48). This ring flow is the cause of sedimentation; without it, sedimentation would not be conceivable. The ring flow is at the same time a means of destroying energy so that the groins play their part in river regulation not only by contracting the river course, but also by forcing eddied flow and thereby creating greater friction.

In back of the pool between groins, where deposition has not

taken place, a knoll is formed adjoining the upstream side of the lower groin (Fig. 49). The sediment wanders through the pool to the knoll. Engels found by experiment that both the pool and the knoll greatly influence sedimentation. Pools may become dangerous if they lengthen to the extent that they join each other. If they become joined, a secondary channel is formed in front of the groin heads. The higher water velocity in the secondary channel causes a "suction" upon ships and creates the danger of their being driven against the groin head. This danger can be forestalled by stabilization of the bed.

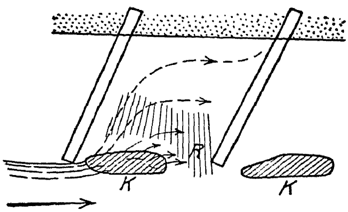


Fig. 49. Scour and ridge formation in groin fields

Engels gives the result of his groin experiments as follows:

At LW and for a small amount of flow over the groins, the channel side and downstream part of the groin field receives sediment; at HW the shore side and upstream part of the groin field becomes filled. Since the latter deposit — insofar as the sediment at the river bottom comes into question — comes entirely from the head of the upper groin, it is not hindered by the knoll, *R*, which is formed during LW and raised during MW stages. The scoured channel, arising along the downstream side of the groin during MW stages, produces capability for sedimentation at the next HW stage. Although harmful to the permanence of the groin body, this channel is useful in connection with sedimentation at high stages.

Groins directed upstream are superior to normal and downstream groins with reference to shore protection as well as sedimentation in the groin fields. Downstream groins endanger the shore most seriously, because water flowing over them normal to the direction of the groins is directed toward the shore. Furthermore, sedimentation is weakest for this type. The scour at the heads is smallest for the downstream groins, but this advantage is more than offset by the disadvantages.

The distance between groins in straight river stretches may not be made greater than the contracted width at LW.

HW sedimentation is aided by the pools at the groin heads.

If the formation of the pools at the groin heads is hindered by stabilizing the river bottom, flat-headed groins are superior to steep-headed groins as far as HW sedimentation is concerned.

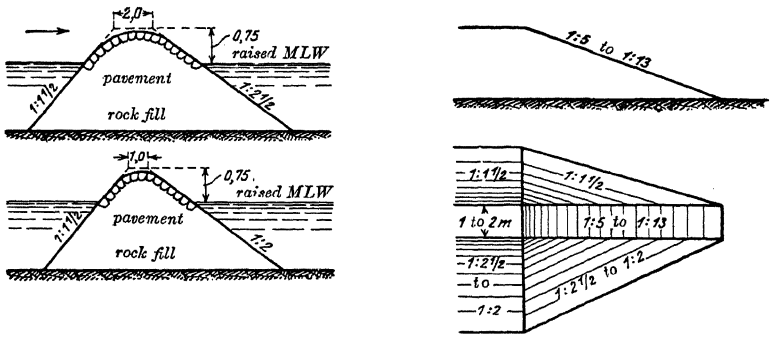
If the development of the pools at the groin head is unhindered, the shape of the head has no influence upon HW sedimentation. Flat-headed groins are to be recommended if the river bottom is stabilized in the vicinity of the heads and if the formation of pools below the heads is hindered.

At present, most river regulation is made for the LW stage, because navigation is most disturbed at this period. For such regulation, groins are constructed from the old shore to the flank at LW, so that the groin

heads lie at LW elevation. The flatter the groin head is feathered out to the river bottom, the more favorable the layout will be for navigation.

2. Construction of Groins

All groins must be tied into the shore far enough so that scour around the end is impossible even at HW. The cut at the shore is called the groin chamber. For large rivers it is as much as 40 m. (131 ft.) long but, in general, 3 to 5 m. (9.84 to 16.40 ft.) suffices. Figs. 50 to 53 show diagrams of rock fill groins. It will be observed that particular care is

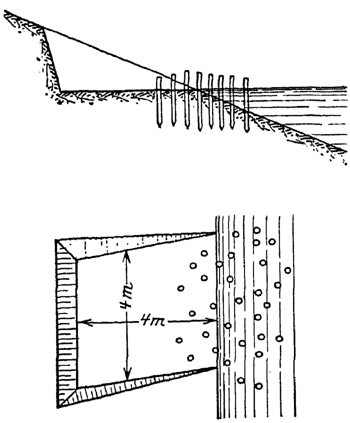


Figs. 50–53. Rock groins used in connection with the regulation of the Weser River
Figs. 50 and 51. Cross-sections
Figs. 52 and 53. Longitudinal section and plan of groin head
Fig. 50. Groin at convex shore
Fig. 51. Groin at concave shore

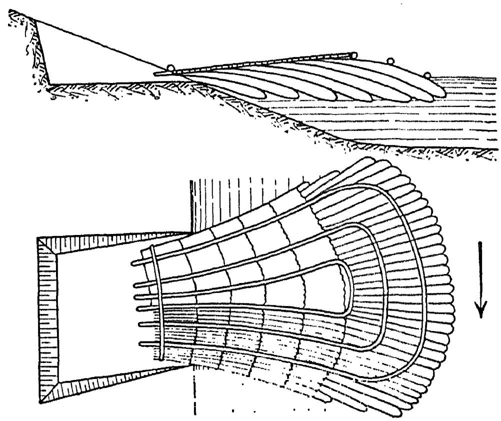
always taken to make the downstream side of the groin especially strong. When the groin is overflowed, a fall occurs at the downstream side, which may easily prove destructive. The inclination of the groin head depends principally on the slope of the river. A steep slope of 1 to 5 was used on the Weser in stretches having a flat gradient. A flat slope of 1 to 15 is used where the gradient is steep and the cross-section is irregular. Paved groins are substantially more permanent than the rock fill. The paving is especially safe at times when the river carries ice, because during the ice period stones can readily be torn from the groin. The paving should extend as deep as practicable. The downstream side is often further protected near the head by fascine rolls. To avoid pools, the groins must be extended at the bottom by further rock fill or wattlework laid on the stream bed. Gravel has also successfully been applied instead of stone. Even sand is used as material in the core, but must be covered by gravel or rock.

Groins made of fascines are generally more difficult to construct than rock-fill groins. Construction begins by excavating the groin

chamber (Figs. 54 and 55). The first layer, 30 cm. (11.8 in.) thick, extends from the chamber (Figs. 56 and 57) and forms the first floating



Figs. 54 and 55. Groin chambers



Figs. 56 and 57. Construction of the first floating layer

layer; the visible projections are only stem ends. Figs. 57 and 58 show the lower half of a single floating layer. The recess is then built back from the front end of the projection, showing only the stems of tree tops on the upper side. This first floating layer is spanned with wire beads or fascine rolls, which are fixed on suitable pegs. The layer is then weighted down and sunk with gravel or sand. Superimposed upon

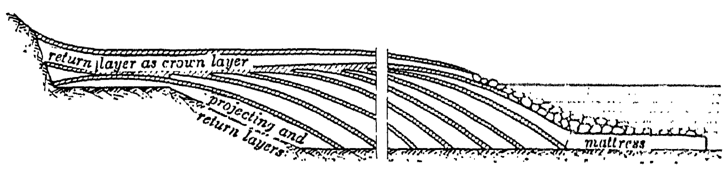


Fig. 58. Completed groin with protection of the foot

this, the next layer is begun so that a section similar to Fig. 56 is obtained. Fig. 56 shows the groin in an unfinished condition; some of the floating layers are already resting upon the bed, the remainder are still suspended. Fig. 58 shows the finished groin. The foot is specially protected by wattlework. The fascines must be well rammed together by hand so that the structure becomes as dense as possible. Stakes may not be driven through the groin into the substrata; hence, the structure remains completely movable.

Before constructing the groin, the river bed must be carefully sounded so that dimensions of individual layers can be accurately

determined.[1] The groins are variously covered with a growth of some sort so that the roots may render the crown especially resistant. If the plants are regularly trimmed, they may become advantageous. How-

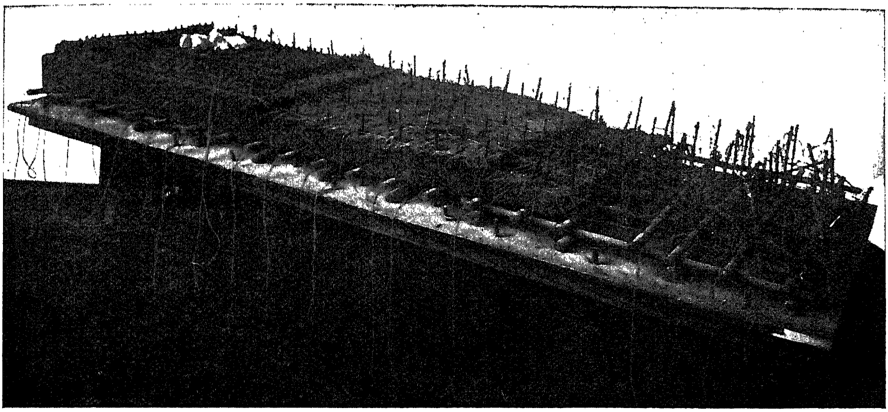


Fig. 59. Fascine groin

ever, Hagen discarded this procedure because it resulted in irregular sedimentation.

3. Construction of Sills

Sills are built of rock fill and fascine construction. The downstream side should invariably be substantially flatter (1:3) than the upstream side (1:1). Sills should obstruct the deep pools; accordingly, they start at the shore and end at the opposite side where the bed again rises. The interval between sills should not be too large — in general, a fraction of the width of the river. An interval of 15 to 20 m. (49.2 to 65.6 ft.) may be considered a good average (Fig. 81).

Substantial improvements are possible in the methods of constructing sills. Engels describes the method used on the Elbe in which coupled, flat-bottomed lighters were used, having a grid floor between them, the intermediate spaces being covered by flaps. If the construction is of fascines, the deepest places should be filled with wattlework, the joining slopes of which are covered with fascine bundles. Short pieces of wattlework are first used and as the hole

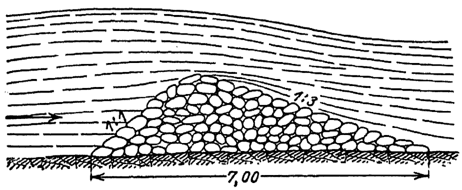
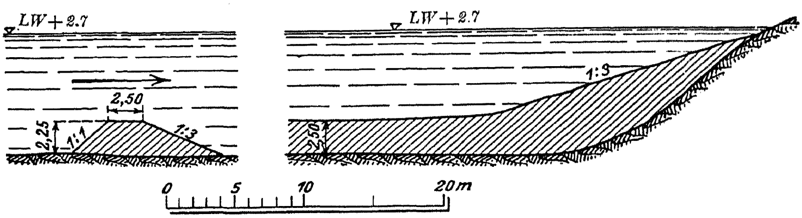


Fig. 60. Rock sill

[1] See *Handbuch der Ingenieurwissenschaften*, Vol. 6, p. 188.

becomes filled, longer ones are successively superimposed. Sills not having intermediate fill with gravel or dredged material have been found unsatisfactory in the development of the Weser. A rise in water level has taken place over every sill. Between them the level sinks if the entire bed between sills is not raised. Sills should, therefore, be considered only as a means of stabilizing a raised bed; the actual raising of the water level must be forced by fill between sills. In the excellent regulation of the Weser (Muttray and Visarius) as many as five sills have been installed in a length equal to the width of the river surface. The top edge of the sill should be 30 to 50 cm. (11.8 to 19.7 in.) deeper



Figs. 61 and 62. Cross-section and longitudinal section of a sill

than the proposed river bottom, so that stones which stand upright in the fill do not extend above the desired river bottom. The sills in the Weser were given a slope of 1:40 from the side toward the center of the river. Entire pools may be obstructed with sills. For this purpose a sill layer may first be filled and then a new series superimposed. Construction can also proceed by building the sills in the form of wide dams to the full height and then subsequently filling the intermediate spaces. The first method of construction is the more economical. (Figs. 60 to 62 show diagrams of sills.)

c. Parallel Works

1. General

When groin fields have been well filled with sediment, protective cover is required for the new shore, and in this connection longitudinal works are built along the new flank. Such longitudinal works may be constructed immediately, without groin construction, in the form of parallel works. If an immediate regulation result is wanted, sedimentation of the groin fields cannot be waited upon. The flowing part of the river must be separated from the part to be filled by constructing longitudinal dams. These parallel works for LW regulation are laid with the crown 20 to 30 cm. (8 to 12 in.) over the governing MLW level. They must be well tied in at the upper end in order to avoid the possi-

bility of scour around the end. As long as the parallel works are not overflowed, the water elevation at the upper end is approximately the same as the stage of the river at the lower end. If a river has a fall of 1:2000 and the parallel works is, say, 500 m. (1,640 ft.) long, then the water elevation in the upper closed off space will be about .25 m. (.82 ft.) lower than the surface of the adjacent river. This difference can be decreased by transverse dams between the parallel works and the old shore, since the parallel works are always somewhat permeable. These transverse structures have a further purpose in that they forestall the formation of deep channels behind parallel works during HW (Fig. 63). Sedimentation occurs very slowly behind parallel works, since the bed load of the stream does not readily move over the structures. On the other hand, good discharge areas for dredged material are created, thus precluding the danger of allowing the sediment to be washed away, as is the case in fields between groins.

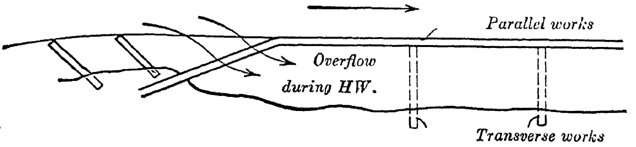


Fig. 63. Transverse structures at parallel works

Sedimentation behind parallel works can be improved by allowing a gap just below a transverse dam, so that the river may enter the field. According to Kreuter, this method has proven successful.

2. Construction of Parallel Works

The best and most permanent structure is obtained by the use of rock fill. Rock embankments can be built of quarry stone from scaffolding or poured from flat-bottomed lighters. The fine material is used in the core; the coarse, in the shell. Paving of the outside is generally desirable for navigable streams, but should be undertaken only after the embankment has well set. The crown width is usually chosen as 1 to 1½ m. (3.28 to 4.92 ft.); the slope on the river side 1:1.5 to 1:1.3. The division of the work into several construction years lessens the total cost. Figs. 64 to 66

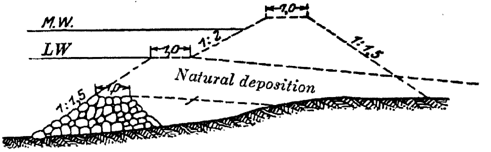


Fig. 64

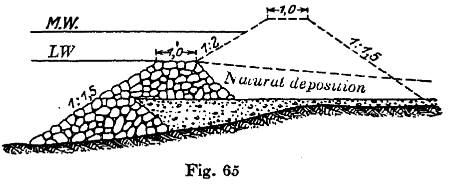


Fig. 65

show how three dams may be constructed one after the other; the second and third rest upon the sediment which has been deposited between construction periods. Sections of a control works built as a unit, with the core and shell resolved into separate parts, are shown in Figs. 67 and 68.

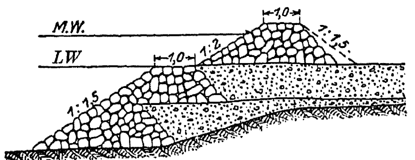


Fig. 66

Figs. 64–66. Normal profile for parallel works purely of stone construction on the middle Danube of Bavaria. The structure is built in three stages, after each of which natural deposition takes place.

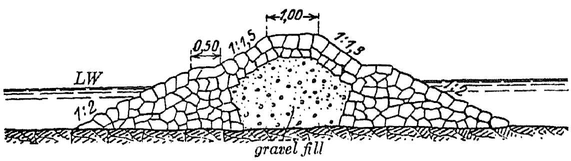


Fig. 67. Parallel works on the Main showing core and shell

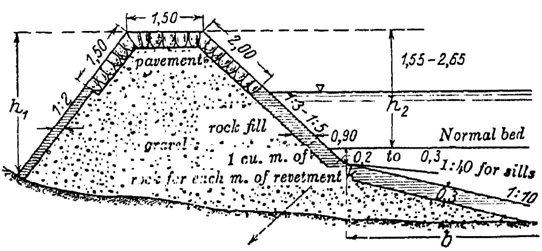


Fig. 68. Type of construction used in parallel works for regulation of the Weser

For brush construction, the most essential unit is the wattlework. In such construction, careful consideration must be given as to whether serious sinking of the LW level will take place. If it does, early rotting of the brush material is to be expected. If the shore is well developed and protected by that time, this decaying will do no harm; there will simply be somewhat greater settlements than would occur without the parallel works.

Transverse structures between the parallel works and the shore are generally built after the parallel works have been completed. The crown of the transverse structures is made either the same height as the parallel works at their juncture, or extends 20 to 25 cm. (7.9 to 9.8 in.) higher and slopes upward toward the old shore at an inclination of 1:100 to 1:30. The transverse structures, just as groins, must be well tied in to the shore, but may not tower above the foreland of the river. No wattlework is necessary where small depths are encountered. Workers may be allowed to carry on the construction in rubber suits when the depth is .8 m. (2.62 ft.) or less, and the work can be performed as in the dry.

d. Movable Restriction Works

Dip-trees serve the purpose of creating quiet water and, therefore, sedimentation in the river. They consist of small deciduous trees anchored by chains to concrete blocks. The tree and stone should not be tied too rigidly together; a movable connection allows the tree to take a position which suits it best. Dip-trees are a cheap means of regulation, especially for obstructing large river widenings and pools. They often become so well covered with sediment that they must remain in the new strata. The deposits created in this manner must be covered by rock.

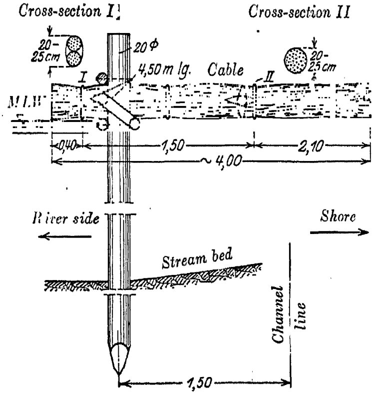


Fig. 69

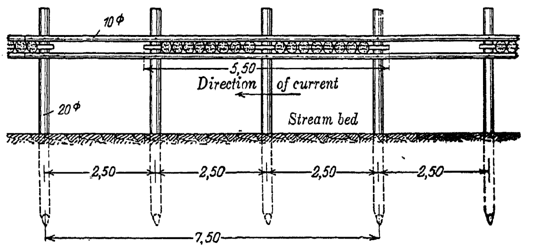


Fig 70

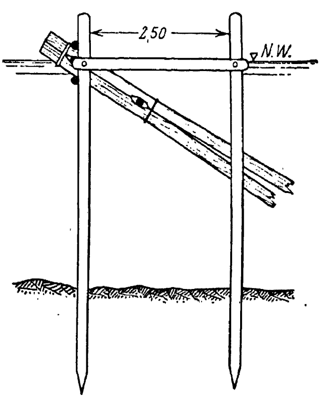


Fig. 71

Figs. 69 and 70 show arrangement for the single line of piles and Fig. 71 for a double line

Figs. 69–71. Wolf pendants

Wolf's pendulous construction makes use of fascine layers. Fascine bundles 30 cm. (11.8 in.) thick are fastened at the stem end into sections, so that the single bush end retains two stem ends. Two poles 8 to 12 cm. (3.2 to 4.7 in.) thick are then placed through these bundles, thereby forming sheets. The pole nearest the end is the hinge pole; the other, the stiffening pole. The sheets (Figs. 69 to 71) are supported at 2.5 m. (8.20 ft.) intervals; the fascines are made in lengths up to 4 m. (13.1 ft.). The supporting piles are driven parallel to the direction of flow at 2.5 m. (8.20 ft.) interval and about 3 to 4 m. (9.8 to 13.1 ft.) from the shore. The piles are .2 to .25 m. (.65 to .82 ft.) thick and extend

only a small amount above MW so as not to be damaged by ice and debris carried down the river at high stages. The pendulous sheets are fastened to the piles with wire at MW elevation, the bush ends being toward the shore (Fig. 72). Alternate fields of 5 m. (16.4 ft.) length are obstructed, and intermediate fields left open. Thereby a continually suspended, very rough, mat chain (a movable bed, so to speak) is obtained in front of the flank of the river. This causes a marked retardation of the water. Since this chain contains regular gaps, the sediment wanders behind the pendulous sheets and then comes to rest. In a short time, enough deposition takes place behind and under the sheets to form a new shore line at the river side of the structure. In case of strong current, the supporting columns are strengthened by a second series of piles driven nearer the shore and tied to the first series by transverse timbers (Fig. 71).

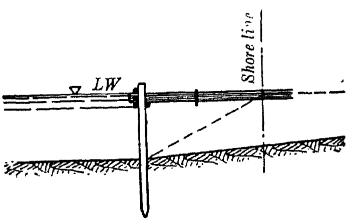


Fig. 72

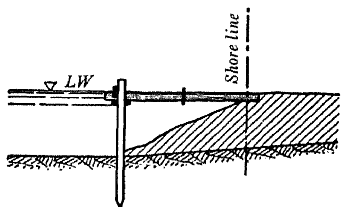


Fig. 73

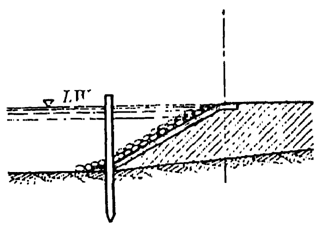


Fig. 74

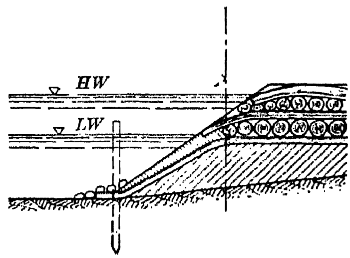


Fig. 75

Figs. 72–75. Wolf pendants, showing the progress of sedimentation and the final form of shore protection.

Kreuter recommends not to stabilize the deposit too soon, but to await a certain degree of natural stability. When the supporting columns are correctly located, sedimentation does not reach quite to the flank, thus allowing space for stabilizing the new slope. Figs. 72 to 75 indicate the intermediate conditions and the final condition. Fig. 74 indicates the deposit held in place by fascines and rip-rap. In Fig. 75 the entire shore is developed; the bed in front of the new shore has deepened since the channel was contracted.

In addition to pendulous sheets of brush on rigid series of piles, proposals for pendants on floating poles have been made by Moeller, Leiner, and others. Doubtless such methods may be successfully developed, but up to the present the best results have been obtained with the Wolf pendulous sheets. They are a great improvement over the rigid construction methods, but require very accurate knowledge of the river and precise observations during construction. Of late (1926) transverse wire groins and backwater rafts with parallel floating damming walls have been used experimentally on the Weser and Aller rivers. The result is said to be good.

e. Shore and Bed Covering

Protective coverings for shores are either completely constructed immediately or a portion of the work is left to the river. In the first case rock is placed on the smoothed slope and frequently low embankments are set in front of the shore to protect the foot of the cover. Figs. 76 to 80 show this type of construction.

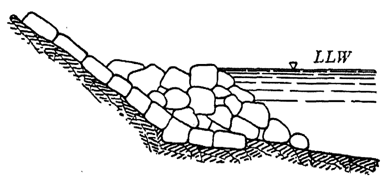


Fig. 76. Simple shore protection consisting of low rock-fill dams in front of the shore.

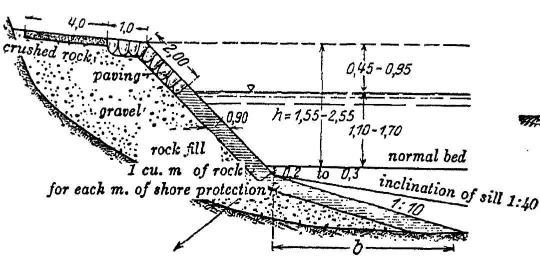
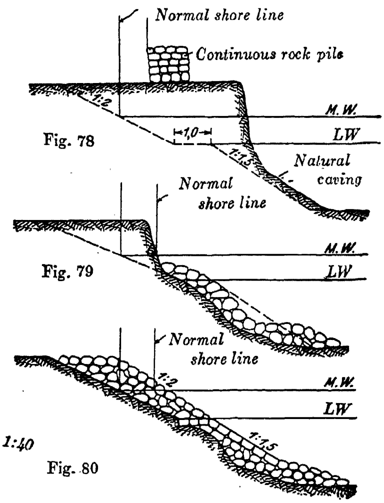


Fig. 77. Type of construction used for shore cover in connection with regulation of the Weser



Figs. 78–80. Shore protection by means of rip-rap in which rock falls into the river as a result of natural erosion and caving of the bank

Brush or fascines are used as abattis or retards which, however, usually require a further cover of rock. Fascine cover works are not being used on the Weser at present; they are replaced by stone or rock fill. Heavy protective construction differs substantially from the construction of parallel works. Fig. 77 shows a diagram of the shore protection of the Weser.

In case recourse is to be taken in the assistance of the river, continuous heaps of rock are distributed upon the shore. The river gnaws its way up to these rocks and causes them to roll down the new slope and cover it, thereby forming protection from further erosion. The first cover may be further developed to a more permanent construction during the following spring and summer. Examples of such developments are indicated in Figs. 78 to 80.

Bed coverings have been treated in connection with restriction works. The type of construction, of course, whether wattlework, fascine rolls, or rock and gravel fill, depends upon the material at hand. Wattlework which is continuously submerged and well weighted down by rock may be considered as practically permanent if the river does not carry coarse detritus. Rock protection, however, is usually easier to apply and also allows for necessary changes in the river bed.

f. Evaluation of Individual Construction Methods, Particularly Comparisons between Groins and Parallel Works

Much discussion has arisen as to whether the use of groins or parallel works provides the correct method of river development. As is usual in disputes, the correct solution lies between the two. Where the new flank lies at a long distance from the shore, groins are ordinarily most suitable; where it lies close to the old shore, parallel works should be used. At the convex shore groins should generally be used, while at the concave shore parallel works are advisable. An arrangement similar to Fig. 81 results. This layout also has the advantage that the groin heads at the convex side produce a very desirable deepening of the section, thereby reducing the irregularity in form of successive sections.

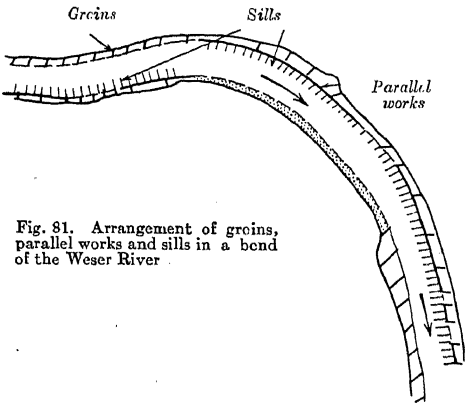


Fig. 81. Arrangement of groins, parallel works and sills in a bend of the Weser River

Parallel works are superior to groins in their effect on navigation. The movement of water along parallel works is uniform and freer from eddies than in front of groin heads. With the construction of parallel works, success of regulation follows immediately. From the standpoint of design, groins are superior because they may be lengthened or shortened. Mistakes in design can be cor-

rected if groins are used, but not with parallel works, since for the latter the entire structure must be replaced by another, and if the parallel works need to be moved toward the shore, the first longitudinal structure must even be removed. Economically, the groins are superior, even if the design of the parallel works is satisfactory. Groins are cheaper to install and because of the more rapid deposition, they are cheaper to maintain.

Movable structures have proven to be a very economical aid to regulation, as they are particularly well adapted to variation in flow. Design errors are of still less significance in these than in groins. Pendulous structures and the dip-tree method are doubtless in a position to offer competition successfully to the older construction methods.

PART TWO

RIVER MOUTHS AND THEIR TREATMENT

A. GENERAL

a. Purpose of the Correction of River Mouths

River mouths are the natural entrances for transportation from the sea to the interior. The purpose of correction is to make the river mouth passable for sea-going vessels and capable of discharging the flood flow.

The farther inland goods can be taken without reloading, the lower transportation cost will be. The task of forming the river mouth to allow deep draft ships to reach harbors lying far inland is one of great economic significance. Thus, the civil engineer is confronted with the problem of determining what depth is attainable, and how it can be most cheaply provided.

The correction may cost millions, but its usefulness is invariably worth much more than merely the interest and liquidation of the capital. Large harbor cities may grow up far inland and the development of older cities may be greatly stimulated as a result of the successful correction of river mouths. Glasgow and Bremen are typical examples of such development. Industrial districts have been added to these cities, which are now capable of competing with foreign industries because of cheaper access to the raw materials. A gain accrues to the countries having harbor cities; part of the gain results from a saving on transport costs. In any case, the indirect benefit is especially significant and commends river correction not only for countries having large industries, but also for lands where these are still to be created.

According to the testimony of English experts, England owes much of its wealth to rapid progress in the correction of its larger rivers. The sharp upward trend of Germany's commerce before the World War would be inconceivable without the correction of the mouths of the Rhine, Weser, Elbe, and other German rivers.

The advantages to agriculture are also important because the water which would otherwise be held back during long continued HW stages in the upper courses may be conducted off more rapidly, and the dangers of overflow lessened if the river mouths are properly corrected.

b. Concept and Classification of River Mouths

In the mouth districts of rivers many phenomena occur having but little in common with occurrences in the upper river. The treatment in Part One of this book is applicable to river mouths insofar as there is similarity to the upper river. The differences overbalance the similarities to such an extent, however, that the treatment of mouths makes special consideration necessary.

There is no definite limit of the mouth seaward. From a hydraulic standpoint, it is suitable to designate the limit by the location, first, at which all bordering shores of the mainland and all restraints to the river flow by submerged bars are absent, and, second, where the current of the river is negligible in comparison to the movement of the sea.

These two locations approximately coincide although not exactly. This is not important since neither of the two locations have fixed positions, but depend upon the river discharge rate and the variation in stage of the sea. The location at which the current of the river is negligible in comparison to the sea motion, for example, may wander back and forth as much as 10 km. (6.2 mi.) or more for the same river, depending upon the rate of discharge of the river.

The landward boundary of the mouth does not remain fixed any more than the seaward boundary. In delta land, the point at which the river divides itself into its principal discharge arms may be considered the beginning of the mouth. There is no such limit in the case of tidal funnels. The limit of the tidal district, at which tidal phenomena cease and which often lies 100 km. (62 mi.) inland, cannot well be called the upper end of the mouth because the largest portion of this stretch is principally of an inland character. Hence the upper boundary is considered at the location where the stream broadens greatly and assumes the character of a bay with brackish water.

In rivers with strong ebb and tide, the entire tidal district should be treated in conjunction with the mouth. To be sure, the appearance of the upper tidal district is more like the upper river than the mouth; nevertheless, the method of correction is fundamentally very different.

River mouths are grouped in two ways: first, according to the kind and size of the river and the quantity of sediment transported; second, according to the nature of the sea into which it discharges; that is, according to whether the tides are strong or weak in comparison to the size and character of the river.

If a river carries so much sediment that neither the tides nor coastal currents are capable of conducting it out of and away from the mouth, and if the slope of the sea bottom at the river mouth is mild and the water shallow, then the sediment will deposit within the mouth, decrease

the water depth, and finally build up new land. As a result, the river will become split into several mouths which spread from each other like so many rays. Every new mouth is active in building up land, some forming secondary arms. This fan-type mouth with the newly formed land lying between the river branches is called delta land or simply delta.

If the deposit does not reach above the surface of the water, it is called under-sea delta. In case the deposition is not heavy enough to cause the formation of several arms at the mouth, the sedimentation is identified as bar-building and results in land-ridges under water. These bars usually have a persistent position transverse to the river similar to the bars in the upper river. They reduce the depth of the entrance. Where the river flows through a closed-off bay — as is the case of the Oder, Memel, and formerly the Nogat — a delta develops at the entrance to the bay and, at the exit, one or more mouths without a delta arise. This form, having two successive mouths after each other, is called an indirect mouth. A mouth which does not go through a *haff*, or lagoon, is called a direct mouth. The Elbe, Rhine, Nile, Mississippi, Danube, and others have direct mouths.

In case the amount of sediment in the river is very small or is transported far enough by tides or strong coastal currents so that bars form only in extreme cases, unified mouths arise which widen and deepen toward the sea. They are either a valley-like natural continuation of the river valley into the sea, or a result of scour by ebb and tide. Such mouths, when occurring on seas subject to tides, as is most frequently the case, are called tidal funnels or estuaries.

All intermediate combinations occur between the limiting forms described. It is to be strongly emphasized, however, that the difference between weak or strong tides is indefinite as a basis for distinguishing mouths. The effect of tides is dependent upon the size of the river and the amount of sediment carried in comparison to the tidal range. Tidal effect is particularly significant on flatly inclined coasts.

Even large tides do not hinder delta formations in the case of giant rivers, while relatively small tides overcome such formations in small streams. The larger the river, the more readily a delta forms; the greater the tide, the more readily a tidal funnel is created. However, delta and tidal funnels do not affect each other adversely. The giant river builds its delta, the tides build the individual delta-arms into deep tidal funnels. Such occurrence has been observed on the mouth of the Rhine,[1] Orinoco, Ganges, and other rivers. Conditions are different for tideless deltas,

[1] Holland was called a part of France by Napoleon I because of being an alluvial formation of the Rhine.

such as those of the Nile and Danube. They do not possess natural funnels; their mouths are shallow and disturbed by high bars.

Since the presence or absence of strong tidal scour is of such great significance in the correction of river mouths, rivers should not be classified for further discussion according to whether they are with or without delta land; instead they should be classified either as

1. Rivers having mouths subject to weak tides, or
2. Rivers having mouths subject to strong tides.

B. MOUTHS SUBJECT TO WEAK TIDES

a. Relations of Current

For weak tides two cases come into consideration; namely, direct flow into the sea (direct mouth), and flow through a closed-off bay or lagoon (indirect mouth). The formation of delta land will occur in a similar manner for either the direct mouth subject to weak tides or the indirect mouth. In the latter case, the delta land forms in the lagoon. A difference will occur for a direct mouth if a one-sided coastal stream moves the delta sidewise.

All mouths must accommodate a reversal in the direction of flow due to abrupt rising of the sea. This reversal occurs most frequently at the outer mouth of the lagoon. If the sea rises very quickly, the river is not in a position to bring enough water into the basin so that its rise can keep up with that of the sea. Sea water will flow through the outer mouth until the outer and inner water heights are the same. When the sea ebbs, the collected river water and the river current must flow out of the lagoon, as well as the sea water which has entered. Hence, a strong current is developed, which causes widening and deepening of the lagoon mouth. These flow relations are fixed by the relative water stages and rate of discharge of the river, the surface elevation of the sea, the wind, and the size and capacity of the lagoon. Since these conditions are different for every river, no general rules can be given for the relations of flow at the lagoon mouth. Each individual case must be treated separately.

A reversal of flow occurs less frequently for direct mouths than for the seaward mouth of indirect mouths. For the former, the occurrence is limited to storm tides and then only for short-time intervals. In the Mediterranean Sea, the water usually rises so slowly that the river gradually develops sufficient backwater to prevent reversal of flow even at the mouth.

Taking three water stages, LW, MW, and HW, into account, nine different flow combinations can occur. The strongest seaward current

occurs for HW stage in the river and LW in the sea. The strongest reversal of flow arises with high sea water and LW stage in the river. Reversal of flow can occur with high sea and MW or LW river stage, as also for middle sea-stage and low river-stage, hence in three different cases.

Bar formations develop at the mouths of nearly all rivers. The bar development is generally more pronounced the more sediment the river carries and the more concentrated the salt in the sea. Strong salt solutions force suspended matter to drop more quickly than weaker solutions. Thus, the bars of the rivers entering the Mediterranean Sea are much more rugged than those of the Baltic Sea or in entirely fresh water seas such as the Great Lakes.

In the case of lagoon mouths, both inner and outer banks often occur as a result of the changing direction of flow. When water flows seaward, the coastal stream is disturbed; hence, the sand carried through the mouth comes to rest and builds bars. When water flows into the lagoon, it takes with it the sand carried by the coastal current and lets it fall inside of the lagoon, since here the flow practically ceases. These bars frequently change in length and form.

The depth of the entrance to rivers having weak tides or none at all is determined by the depth of the bars. Nothing is gained by deepening the river for sea ships unless the bars are removed as far seaward as they are obstructive. The first step in the correction of rivers without tides should consist in deepening of the bars at the mouth, if possible to the depth of the remaining mouth. Only after this has been accomplished should there be further deepening of the mouth including the bars.

Corrections of this type can not be accomplished entirely through control structures; the work must invariably be supplemented by dredging. In especially difficult cases, in which the bars lie far out in the sea where the construction of river structures is impossible, dredging is the sole means of relief.

The erection of moles, parallel works, and the like serves to generate a strong, scouring current. A knowledge of the velocities of flow is necessary in the design of such regulating structures. This information, however, is very difficult to ascertain. The rising of the water surface does not invariably denote an increase in the velocity, but may signify absence or reversal of flow. Since the sea often rises so rapidly that simultaneous measurements of the water elevation at different points of the mouth district is impossible, automatic registering gages must be used. The records obtained by means of these gages are used in computing the amount of water which comes from the river, and the amount which comes from the sea. The gage graphs can also be used to determine the

quantity of water that collects in the river channel or lagoon during any period of time. This quantity forms an index to the scouring capacity of the succeeding flow.

b. Correction of Direct Mouths Subject to Weak Tides

In river improvement work, one is concerned only with the method of making the correction; but, for river mouths, in addition to the method of correction, particular concern must be focused on the choice of the arm of the mouth to be improved. If primary consideration is to be given to satisfactorily carrying off high water which is harmful to agriculture, the construction of a high-water bed is the most important. The removal of bars then is of importance only when the bars cause ice jams which threaten inundation of the country. However, if the furthering of waterway transportation is of primary importance, as is the case of most of the middle-size and larger rivers of Europe, then the development of the river for the low and middle water stages together with lowering of the bars is of major importance. A form of bed corresponding to Fig. 82 is recommended for the development of a navigable channel in the upper mouth.

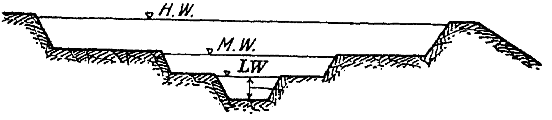


Fig. 82. Cross-section of the bed in the mouth district with special LW, MW and HW beds

In the lower mouth district sand banks must be dealt with. High shores or dikes, and thereby the holding together of the high water, are unnecessary. Fig. 83 shows a plan of the Mississippi mouth, which radiates in four arms, each of which has developed secondary arms. The tidal range here is .4 m. (1.3 ft.) for spring tide, .2 m. (.7 ft.) for neap tide; the river flows into the Gulf of Mexico, a sea in which weak tides occur. The Gulf is characteristically similar to the Mediterranean Sea in this regard.

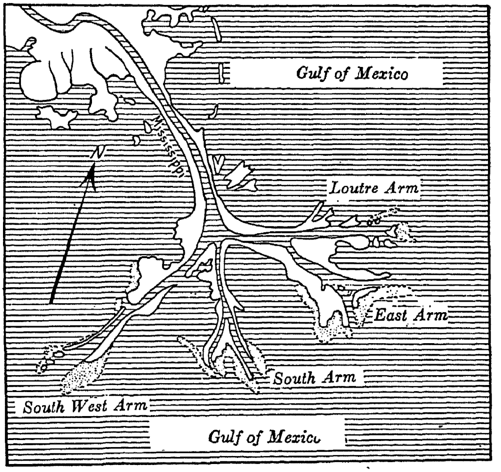


Fig. 83. Gulf of Mexico

The cross-section for the correction is divided into a HW channel, a MW channel, and a LW channel. The first must be so determined that it can carry the backwater from the sea as well as the river flow without danger to the levees. The MW channel is required to carry the most numerous and enduring water stages. It must lead into the deep water; that is, corresponding to the size of the ships, a depth of 8 or 12m. (26 or 39 ft.) under MW. However, inasmuch as the side boundaries for the MW height are absent in the open sea, it is necessary to border the channel by moles which tower to an elevation over high water in order to hold the stream together and to provide a safe route for navigation. The moles cause the concentrated HW to exert a strong scouring action periodically and thereby lessen the dredging requirements. The LW channel must also reach as far as the bars in order to hinder growth of the bars during LW periods.

The first step in the construction procedure is to enclose the MW by groins directed upstream or by longitudinal control works. Longitudinal works are used in the upper mouth district where groins would be too short. (Compare with Part Two, River Control.) In the district of the lower mouth, that is, in the stretch where shores are absent and only submerged sand banks border the river up to the time of construction, the MW channel must be enclosed entirely by longitudinal embankments. They can be erected in the usual manner, using mattresses weighted down with rock.

The LW channel is developed in the same manner as the HW channel. However, the low brushwood dikes required for LW longitudinal works must be joined to the MW embankments by transverse structures; otherwise, flow could take place along the back of the LW embankments. For best results the space behind the embankments should be filled with dredged ground. At some points, principally at bends, it will suffice to construct an embankment on one side only. In the lower district of the mouth, the wave impact incurred makes the construction of lighter embankments impossible. Here, permanent moles reaching to above HW must be constructed and usually prolonged into the deep water. If the moles are extended as far as the deep sea water, a long period of time will elapse before sedimentation obstructs the entrance to the river. The moles should then be lengthened.

The moles serve as a continuation of the MW channel, but must also allow the high water to pass without causing appreciable backwater. Between them the LW bed is to be carried forward by constructing low, rock-fill embankments. Preferably the LW longitudinal structure should be constructed on only one side, the mole being used for the other side.

In case a strong sea motion occurs in front of the mouth, the longi-

tudinal moles will conduct the waves undiminished in strength far into the river. If the river forms a waterway, this artificially inward conducted sea motion may have harmful effects.

The following guiding principles deserve consideration in connection with choosing the arm which should be corrected. They are based upon the experience gained from the corrections already carried out in the interest of navigation.

a. Guiding Principles for the Regulation of a Delta Mouth in the Interest of Navigation.

1. For larger streams the smallest navigable arm possible should be corrected for the following reasons. It discharges the smallest amount of water and, therefore, the least sediment. The growth of the delta and the bar formation will be the smallest at its mouth. Since the quantity of water is small, the current will subside sooner after the flow reaches the sea. The sand must come to rest sooner, causing the bars to lie nearer the shore than in the case of the large arms. The guide dams which must lead to the bars will be correspondingly shorter and cheaper. *The smaller the arm for sufficient navigability, the cheaper all work will be, and the lower the toll charges; hence, the more economical the layout, and the greater the traffic.*

2. The quantity of flow in the corrected arm should not be increased by closing other arms. Such closure would increase the discharge in the corrected arm, and also the quantity of detritus. The quantity of detritus in the individual arms is a function of the rate of water discharge. In order to decrease the size of the bars, the quantity of water must increase relatively much more rapidly than the detritus. By closing arms which are not to be made navigable, the quantity of water and detritus would increase simultaneously; the bar in front of the open arm would become higher, and, since the periodic turning point of the current would move farther outward, the bar would also move farther out into the sea. The delta would grow more quickly in front of this arm than before, and a much greater elongation of the moles would be required than if the arms were all left open.

3. If there are several arms, one of which is subject to a strong coastal current and the others subject to weaker currents, then the first should be chosen.

4. In case a middle arm discharges into a steeply sloping seashore, it is to be given preference over a smaller arm which discharges into shallow water.

5. If there is no coastal current and further, if the principal wind is directly toward the coast, it is advantageous to create a new mouth which empties at a considerable distance to one side of the delta. It

must be provided with a lock at the river in order to hold back the sediment.

β. Guiding Principles for Corrections in the Interest of Agriculture.

The discharge of ice and high water without harm to the adjacent land overbalances all other requirements as far as agriculture is concerned. Some of the preceding principles should be observed; however, in this case it is invariably preferable to improve a main arm so as to obtain as steep a slope as possible into the sea. Frequently a better solution is obtained by creating a new short mouth. Small arms often must be closed. In the case of high shore lands, the LW stage should be raised, if possible, so as to cause the ground-water level to rise, but this change must not decrease the safety against high water.

c. Examples of Correction of Direct Mouths

Noteworthy examples of the improvement of direct mouths include the correction of the Mississippi, the Danube, the Rhone, and the Vistula.

1. The Danube Mouth

Above Tultscha (Fig. 84) the Danube possesses a uniform bed with a width of over 500 m. (over 1,640 ft.) and a 15 m. (49 ft.) depth. Hence, it is navigable for sea ships. At highest stages, the river discharges up to 27,000 cu. m. (955,800 cu. ft.) per sec.; at the lowest stages, its flow into the Black Sea decreases to 1,700 cu. m. (60,200 cu. ft.) per sec. The discharge at MHW, MW, and LW is about 9,000, 6,300, and 3,500 cu. m. (318,600, 223,000, and 123,900 cu. ft.) per sec. respectively. In the vicinity of Tultscha, the river divides into its three principal mouth arms: the north Kilia arm, the middle Sulina arm, and the south St. George arm, which respectively carry 66, 6, and 28% of the total discharge. Accordingly, the Sulina mouth discharges 210 cu. m. (7,434 cu. ft.) per sec. at MLW.

While the delta of the Kilia mouth consists of 12 very shallow arms and moves seaward at the rate of 90 m. (295 ft.) annually, causing the formation of a long stretch of shallow water beyond the end of the shore, the small Sulina mouth consisted of an arm having a bar of some 1,300 m. (4,265 ft.) from the shore and deep sea thereafter. The St. George mouth possessed two delta arms with a bar extending some 2,600 m. (8,530 ft.) from the shore with deep sea thereafter. Thus, none of the delta arms were accessible to boats from the sea. The disadvantages of the Kilia mouth, that is, many shallow delta arms, shallow sea, and rapid forward movement of the delta, completely excluded it from consideration for

correction. The choice rested between the other two mouths, both of which had certain advantages.

A European commission, called together in 1856, held that the St. George arm, because of greater depth and the proximity of its mouth to the Bosporus, was better than the Sulina mouth. The higher costs implicated, due to the larger dimensions of the St. George arm and its longer bar, resulted in giving preference to the Sulina mouth for the immediate future.

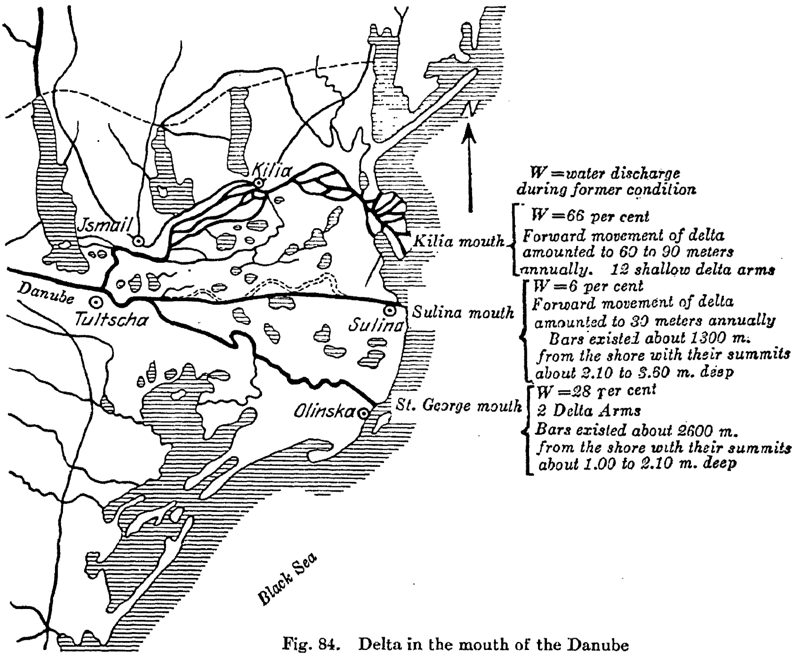


Fig. 84. Delta in the mouth of the Danube

The Sulina mouth was first lengthened with light rock embankments, containing a longitudinal line of sheet piling in the center, as far as the deep water (−5.5 m.) (−18 ft.), where it had a width of 180 m. (590 ft.). After the completion of these dams in the year 1861, the high water of the Danube washed out the bar and created a navigable depth of almost 5 m. (16 ft.) compared to the former 2.1 to 3.6 m. (6.9 to 11.8 ft.). In consequence of this success, the correction of the St. George mouth was dropped from consideration and the moles of the Sulina mouth were further developed and lengthened by the construction of rock-fill embankments which were strengthened by placing large stone blocks on the crown. By 1910 the navigable depth of the pass was increased by the

operation of two sea dredges and the action of the current to 7.3m. (23.9 ft.). The navigable depth fell below 6.4 m. (20.9 ft.) only six days of the year (July and August). Sedimentary deposits occur completely south of the mouth. For a time the shore moved forward in the direction of the moles at the rate of 54 m. (177.2 ft.) per annum (until 1861). However, the rate of displacement has decreased until now it is only 4 m. (13 ft.) annually. The success of the correction is primarily due to the fact that the sediment from the Sulina arm is now transported into the deep sea. It is believed that the rapid forward movement of the Kilia mouth will later require greatly lengthening the Sulina moles.

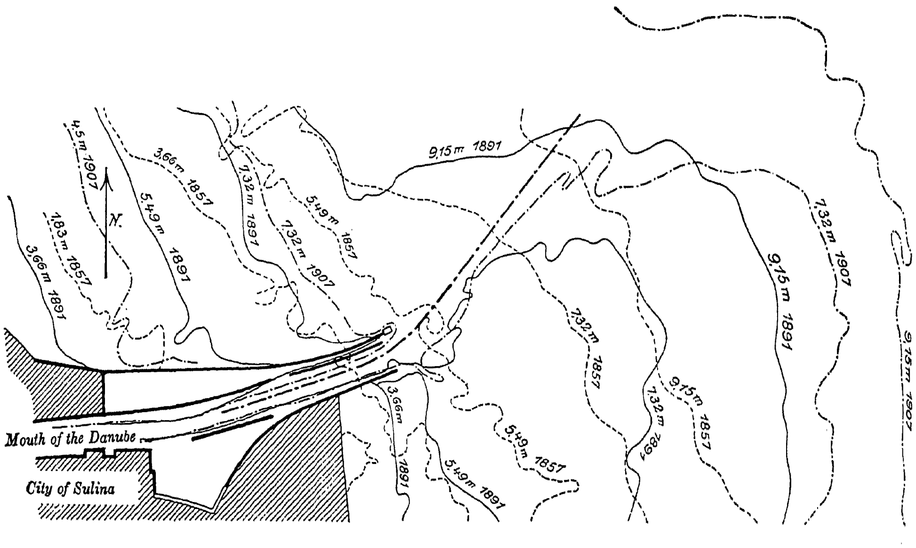


Fig. 85. The mouth of the Danube at Sulina

2. The Vistula Mouth[1]

For many generations the Vistula (Fig. 86) had three principal discharge arms. One of these, the Danzig Vistula, flowed along the coast toward the west with its mouth at Neufahrwasser; a second, the Elbing Vistula, flowed eastward adjacent to the coast to the Fresh Haff; and a third, the Nogat, branched off further up the stream at Dirschau and also emptied into the Fresh Haff. The area between Montauer point and the coast of the Baltic, including the cities of Danzig and Elbing, is delta land formed by the deposits of the Vistula. It is particularly evident that the delta mouth of Nogat and the Elbing Vistula, as well as the narrow peninsula which closes off the Fresh Haff at the north, arose

[1] Weissger, *Z. Bauw.* 1923, p. 39; Salfeld, *Z. Bauw.* 1919, p. 535.

through sedimentation from the Vistula. The Vistula sand was transported eastward by the coast stream, causing older islands of diluvial character to be gradually joined together (Figs. 89 to 96). The average rate of discharge of the Vistula is about 1,000 cu. m. (about 35,400 cu. ft.) per sec.; that is, approximately 30 billion cu. m. (approximately 24 million acre ft.) annually. This water transports over five million cu. m.

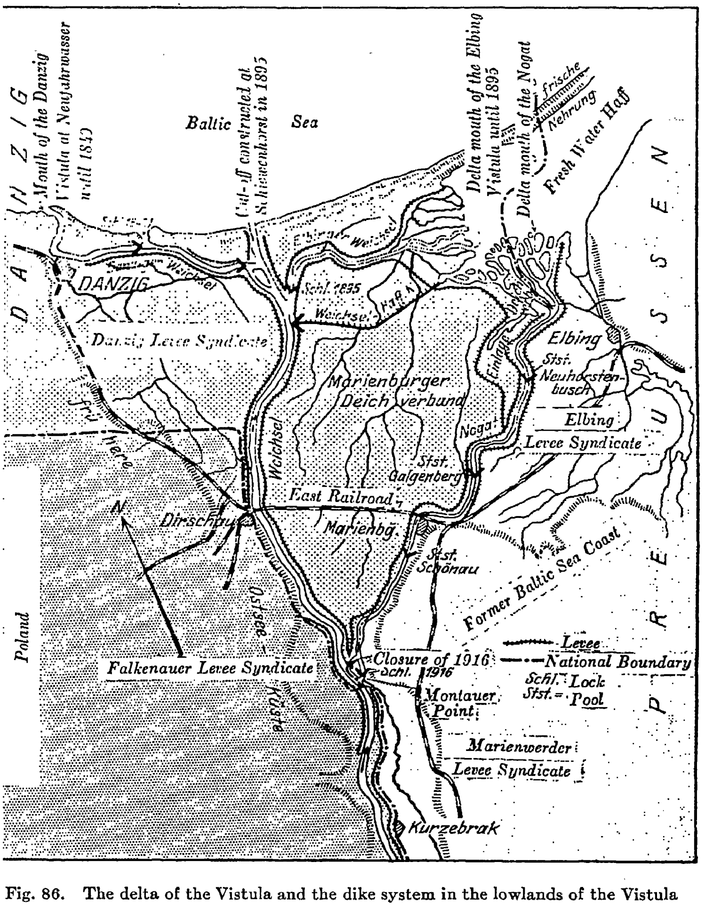
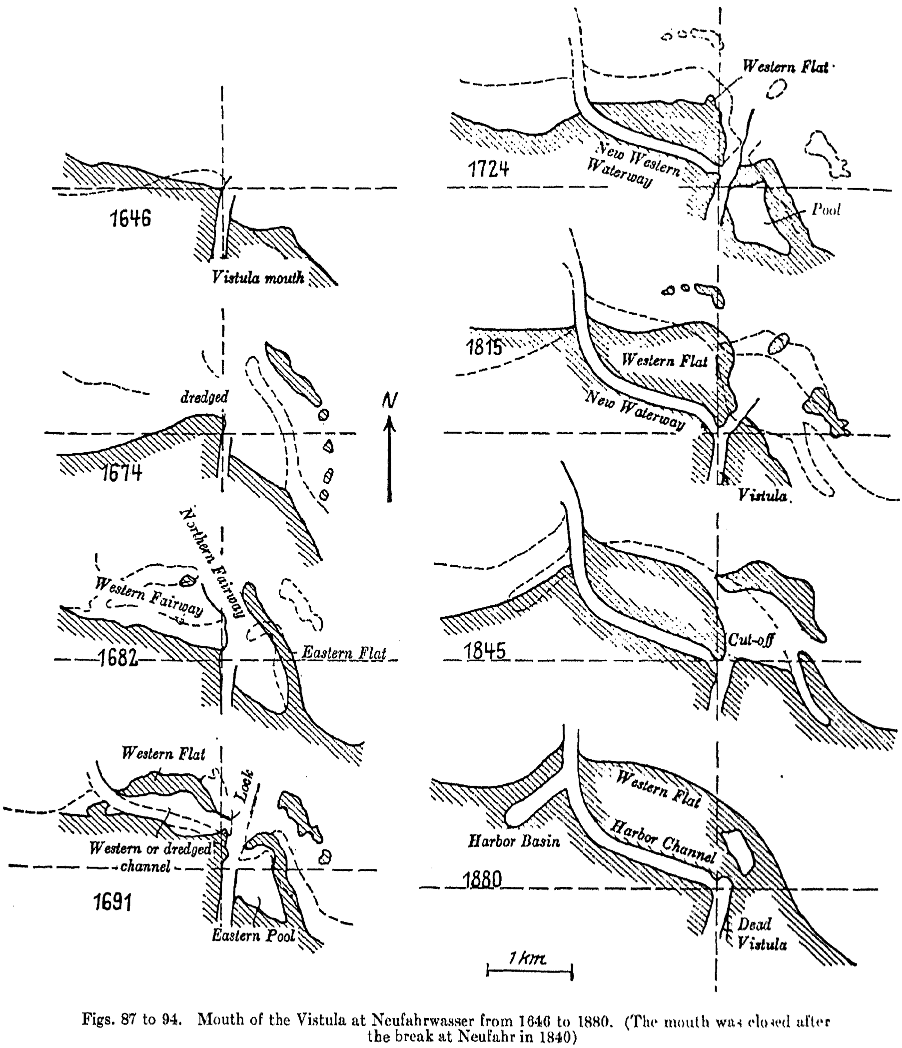


Fig. 86. The delta of the Vistula and the dike system in the lowlands of the Vistula

(over 6.5 million cu. yds.) of sediment. In the early ages the Elbing Vistula was one of the main arms. Later, in the Middle Ages, the Nogat was artificially developed in the interests of navigation, so that the principal discharge was into the Fresh Haff. The deposition in the Fresh Haff is said to have caused the formation of some 13 hectors (32 acres) of land annually. The Danzig arm was newly formed in historic times as a result

of embankments built by the Order Knights. The natural development of the Vistula was toward the east to the Fresh Haff.

Until 1853 the Vistula was a river with principally indirect discharge,



Figs. 87 to 94. Mouth of the Vistula at Neufahrwasser from 1646 to 1880. (The mouth was closed after the break at Neufahr in 1840)

and from the year 1915 was a river with direct discharge up to two-thirds of its total flow. At present it has only a direct mouth. The earlier consideration of the scouring action at the lagoon (Fresh Haff) outer mouth

required that the Nogat be held open. The development of dredges then made the flow of large quantities of water from the Nogat into the lagoon dispensable, so that the Nogat was closed off and canalized. The solution of the problem for bettering the mouth was solved very differently from the manner used in the Danube, which makes the development of the Vistula mouth of special interest.

The conditions of the Vistula have been unfavorable since ancient times. Numerous breaks toward the Baltic Sea have occurred because of ice blockades; the break at Neufahr was caused in this way in the year 1840, disconnecting the entrance of the Vistula from Neufahrwasser. The break was of particular consequence because it shortened the course of the Danzig Vistula 15 km. (9.3 mi.). Considerable increase in fall resulted in this arm, so that the relative discharge increased greatly while the Elbing Vistula discharge decreased. The water depth of the latter decreased to such an amount that it became unusable for navigation and was replaced by the Vistula-Haff canal. The Elbing Vistula, however, was not closed off. The new mouth at Neufahr was retained, and the arm to Neufahrwasser was tied in with a lock. It will be observed that the development to this point was an entirely natural one. The principal discharge flowed through the main arm, the Nogat. The mouth at Neufahr showed little change. The land formation was particularly regular so that the mouth moved forward into the sea as a concentrated arm (compare Figs. 87 to 94). The lower Vistula itself, in contrast to the Rhine, Danube, etc., is not navigable for sea ships. The important harbor cities, Danzig and Elbing, lie directly on the coast and were accessible. There was no necessity of making the Vistula itself navigable for sea ships to points other than these harbor cities. Work was entirely in the interest of a land development and therefore proceeded by the application of other fundamentals than were later used for the Danube.

A radical change took place in the Vistula in the year 1853. The Nogat branch was displaced and a shorter Nogat canal was built below Montauer point. The purpose of this was to so regulate the discharge of the Vistula that only one-third flowed to the Nogat and two-thirds through the Danzig Vistula. The significant step that was taken toward the development of the Vistula mouth was not the closing off of the Elbing Vistula, but the change in quantity of discharge, because the Danzig Vistula was now made the principal sediment-carrying branch. The mouth at Neufahr, which originally possessed very favorable relations, developed island formations and became wild. Fig. 96 shows the confusion for the year 1871; four arms were developing at Neufahr. The deposition was so intensive that in the fifty-year period before 1895 as much new land was developed as had been built up at Neufahr

in the five hundred years before the construction of the Nogat by-pass canal. The regulation which was later (1856) outlined for the develop-

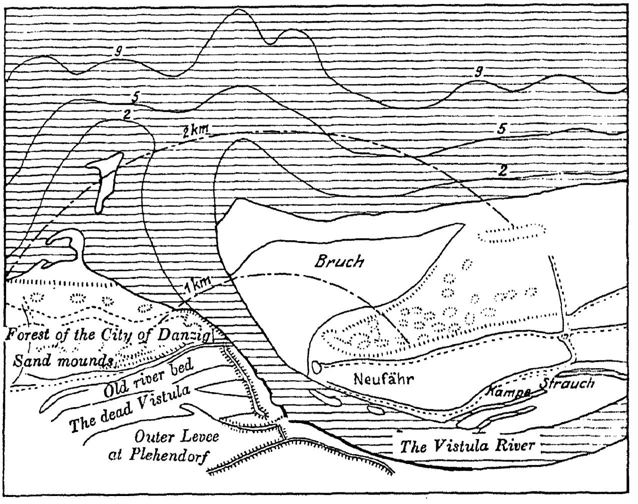


Fig. 95. Condition in the year 1852

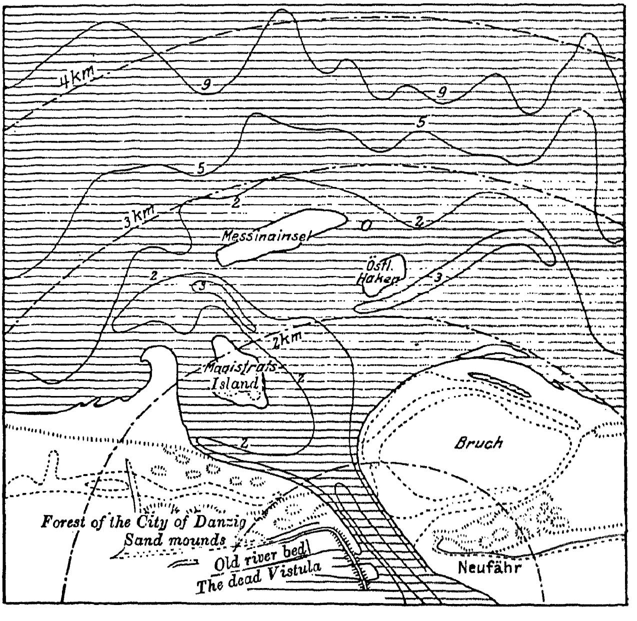
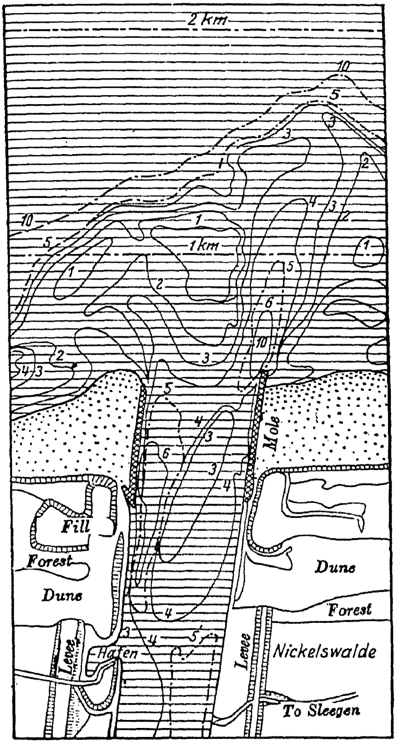


Fig. 96. Condition in 1871

Figs. 95 and 96. Mouth of the Vistula at Neufahr
(The mouth was closed by a break in the year 1840)

ment of the Danube mouth was not applied; the Vistula doubtless showed the correctness of these methods and probably influenced the development of the Danube.

The mouth at Neufahr had to be continually further developed and required the construction of extensive moles far into the sea, but they were not projected forward far enough to cause decided betterment.



Hafen=Harbor

Fig. 97. Condition in 1916

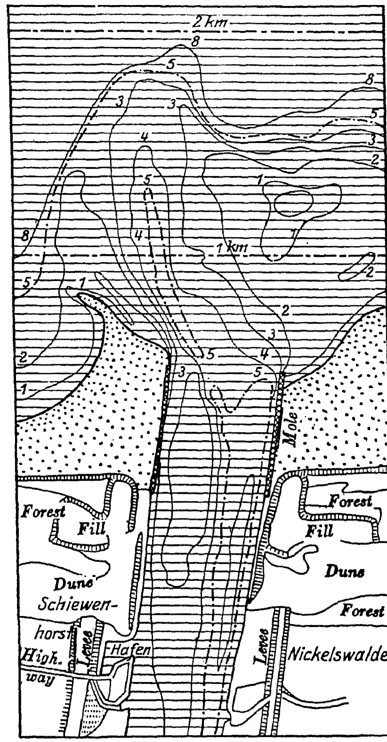


Fig. 98. Condition in 1919

Figs. 97 and 98. Mouth of the Vistula at Schiewenhorst (the cut-off was made in 1895)

The ice jams in the Danzig Vistula remained a continual danger and led finally to the resolution of creating an artificial mouth. The new mouth was formed in the year 1895 by the construction of a cut-off at Schiewenhorst. This was the most successful step in the development of the Vistula mouth up to that time. The closing off of the Elbing Vistula, which occurred at the same time, was consistent with the plan. It was also necessary to cut off the Danzig Vistula by the introduction of a lock. A practical and good solution was thereby obtained in the

interest of navigation, but the development was not yet completed as far as the improvement of land was concerned.

The development of the mouth at Schiewenhorst took place in a manner similar to that at Neufahr. Large sand banks were pushed forward in the mouth. As a result, the mouth wandered back and forth, although a centralized arm could be maintained by continuous dredging.

Since the Nogat mouth does not possess a coastal stream, in accordance with the guiding principles enumerated, the Nogat was finally closed and canalized in the year 1915. Thus, at present, the combined water and detritus masses of the Vistula flow through the new mouth at Schiewenhorst. The result of this last measure was a substantial deepening of the Vistula from the Nogat branch to the mouth, but also a strong bar formation in the mouth itself. Similar to the condition at Neufahr, several arms developed together with the formation of islands (Figs. 97 and 98). Due to the fact that the mouth of the Vistula was centralized into one channel, all sediment was likewise carried to one mouth. Further inference may now be drawn from this unavoidable measure. It will either be attempted to carry the mouth over the neutral zone into the deep water in order to do away with ice blockades as nearly as possible and to maintain the depth of the entrance as continually as possible, or recourse must be sought in continual dredging. Dredging cannot be entirely avoided even if parallel works are constructed. The mouth at Neufahr is now easy to maintain since no sediment must be dealt with.

d. Correction of Indirect Mouths

The mouth of a river in a closed-off bay is improved in a manner based upon the same points of view as described in the preceding section. A noteworthy characteristic of such mouths is the absence of littoral currents. Therefore, if great importance is laid upon the navigability of the river mouth, the installation of a new entrance containing a lock may be suitable. If it is possible to enter the sea at the side of the lagoon by way of the new mouth, this is the best solution. The arm to the lagoon would then be canalized as was done for the Nogat. The present stage of development of the dredge makes the increase of scour by river water in the outer channel appear usually less important.

Correction of fairway: special importance is laid upon retaining the depth of the lagoon mouth (fairway or sea channel). There are many good harbors in closed-off bays, where access depends upon the condition of the fairway.

The aim of correction of the channel consists in getting rid of bars and attaining a narrow, regular course. Since consideration of agricul-

ture is seldom necessary in this connection, the width of the underwater channel is practically fixed by the requirements of navigation. Therefore, in most cases it will be correct to limit the width to the extent that the strongest currents occurring will not hinder navigation. By so doing, the strongest scouring stream will be obtained in both directions. The shore development proceeds as in ordinary river regulation.

The bars which develop from time to time inside or outside of the outer mouth, depending upon the direction of the current, do not receive their sand from the river but from the coastal current. The lagoon actually forms a clarification basin for the river. The strong scour, which occurs intermittently, is usually insufficient to maintain the necessary depth over the bars. In order to eliminate dredging as much as possible, it is recommended that even the weaker currents be concentrated and used for producing scour. For this purpose, low control dams are laid on the inside in such a manner that there is a narrow channel left at the mouth and a widening toward the bay. These dams conduct the incoming stream further into the bay. The velocity will decrease gradually and the sand deposits spread over a large area, avoiding obstructive bars. It will be necessary to lengthen the embankments from time to time and frequently the help of dredges will be required. High moles are usually not constructed on the lagoon-side of the mouth.

Submerged embankments may not be laid on the outside of the mouth because of the danger to navigation. High moles are necessary where an alternating coastal current is encountered and they must be constructed on both the left and right side of the entrance. These moles, to be sure, are most simply constructed at right angles to the shore, but in most cases are laid obliquely in view of waves, currents, and the location of the subchannel fixed thereby. They form a hindrance to the coastal stream, which is crowded further seaward by them. As time goes on, the moles become sanded up, and must be extended into the deep water.

The Memel and Pilau fairways, the Swinne mouth, and the Newa are mentioned as examples. The fairway of the Fresh Haff, called the Pilau sea channel, today the sea end of the Koenigsberg sea canal, is to be described (Fig. 99).

Up to the time of the World War the fairway formed the discharge opening for the Nogat, the east mouth arm of the Vistula, and still is used as such for the Pregel and series of smaller rivers and streams. Today the Nogat is closed off and canalized, thereby being connected with the Vistula only for transportation purposes.

The Nogat discharged about one-third of the HW from the Vistula

(2,700 cu. m. or 95,580 cu. ft. per sec.) into the Fresh Haff and thereby carried more than twice as much water as the Pregel, which discharged a maximum of 1,300 cu. m. (46,020 cu. ft.) per sec. At MW the discharge per second of the Nogat was 360 cu. m. (12,744 cu. ft.) and amounted to about 72 per cent of the fresh water which enters the lagoon. The greatest discharge was 4,630 cu. m. (163,902 cu. ft.) per sec. with a maximum velocity of 1.6 m. (5.2 ft.) per sec. The greatest inflow amounted to about 3,900 cu. m. (138,060 cu. ft.) per sec. Formerly the Pilau sea channel was correctly treated as one of the Vistula mouths. Cutting off of the Nogat signified a radical change of view regarding the value of the scouring current.

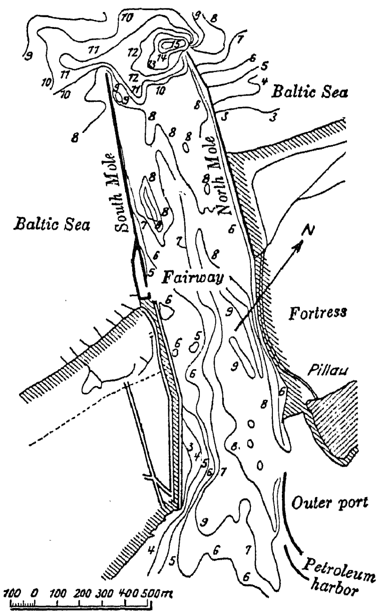


Fig. 99. Pillau Fairway

The two sea moles form the main portion of the fairway improvement. They were constructed to concentrate the current over the outer bars and thereby to deepen the bars. At the shore end, the moles are at a distance of 500 m. (1,640 ft.) from each other and after several elongations now possess an interval of about 360 m. (1,180 ft.) at the ends. They extend to the 8 m. (26 ft.) depth line and are respectively about 1,000 m. (about 3,280 ft.) (north mole) and 1,100 m. (3,608 ft.) (south mole) long.

The moles caused an increase in depth over the bars from 4.7 m. to 5.3 m. (from 15.4 ft. to 17.4 ft.) within a few years. Because of a dike break of the Vistula, a depth of 7.8 m. (25.6 ft.) was attained which diminished, after a time, to approximately 6 m. (approximately 20 ft.). In spite of occasional high velocities in the channel, it is not possible to maintain sufficient water depths by means of the scouring force of the water alone. A large quantity of sand is continually driven from the sea into the mouth. The only way to get rid of this sediment is by dredging. The amount of dredging thereby increased from 7,100 cu. m. (9,100 cu. yds.) in the '60's to 40,000 cu. m. (52,400 cu. yds.) in the '80's. The fact that still more sand did not have to be dredged doubtless rests upon the action of the moles, which cause a large portion of material to be transported into deep water during periods of strong flow.

The part the Nogat played in maintaining the fairway was formerly overestimated. The canalization and the resulting cutting off of the Nogat now lessens the amount of water discharged through the fairway; nevertheless, the channel is not becoming shallower. Eventual losses in scouring action can easily be replaced by an increase in the amount of dredging.

C. MOUTHS WITH STRONG TIDES

a. Behavior of Tidal Waves in Rivers

When two currents of different velocity meet each other in a canal, a rise occurs at the meeting point. However, since the kinetic energy of the two streams which form the water mound are not equal, the mound moves forward in the direction of the current which originally possessed the greater amount of kinetic energy. The two streams of water thus directed against each other become mixed, and a velocity results which corresponds to the difference of the kinetic energy of the two currents. An expression of this occurrence is observable in the development of tidal waves in the mouths of rivers which discharge into seas subject to strong tides.

At the time of rising tide, water flows from the sea toward the continent, and when entering a bay, the current becomes stronger as the bay narrows. Estuaries are extreme cases of such narrowed bays, with the additional characteristic that within an estuary a salt water stream flows toward a fresh water stream. The velocity of the tidal current is small at the beginning, but gradually increases and becomes greater than the river current. Hence, at the beginning of a rising tide, the river water retains its direction, but as the sea continues to rise, the tidal current becomes stronger, retaining its direction though its velocity decreases. Backwater thereby occurs in the mouth, which in turn causes the formation of a wave hill which moves upstream as a slender wave crest. During the time that the tide rises rapidly the crest of the wave lies in the sea and does not progress forward. Near the upper limit of the tidal change, when the rate of rise becomes slower, the crest of the wave wanders upstream.

As long as the tide continues to rise, the mass of water in the river between the wave crest and sea moves upstream. The behavior of this mass of water is similar to that of a ball, which when rolled up a hill must eventually come to rest after a period of time; that is, as soon as its kinetic energy has been consumed. However, in the case of the water, the energy of the moving wave is consumed not only in the constant overcoming of the upward slope and friction but also in backing

up the opposed flow of water from the river. The water of the tidal wave continually mixes with the opposing river current and thereby continually loses velocity.[1]

When the tide begins to ebb, a definite wave hill develops. The tidal hill, which is formed by water quantities impinging against each other from both directions, is a considerable distance up the river by the time the tide begins to recede. At the beginning of the ebb, water must again flow from the river toward the sea. At first this recession can take place only near the mouth, because further upstream the velocity of the tidal current hinders return flow. As the duration of the ebb increases, a ridge sloping toward the sea becomes more fully developed downstream of the tidal hill. Near the crown of this ridge, the flow continues upstream; seaward from the ridge, return flow is in progress. Thus a condition exists in which water continually flows toward the sea from the tidal hill. The tidal wave, therefore, not only loses strength because of the reaction of the river above, but also because of the loss of the water which flows seaward.

When the amount of water flowing seaward from the downstream end of the estuary becomes greater than that flowing in the river itself, the crest may wander still further upstream, its height continually becoming smaller until it finally disappears; that is, it coincides with the usual water stage of the river. The point where it coincides is the ebb limit. The last part of this hill below the tidal limit is simply the backed up river water. The largest portion of this last part of the backwater never ceases to flow downstream; nevertheless, the rate of flow decreases the nearer the crest is approached. At a point a considerable distance above the end position of the crest, tidal flow and river velocity become equal, that is, all current disappears. The last trace of the actual tide reaches to this point, which is the limit of the tidal current.

The occurrence of a tidal wave may be summarized by the description which follows. First a mass of water flows inward as a result of rising tide; this mass loosens itself from the sea and wanders upstream as a wave independent of the sea, entirely due to its kinetic energy. By the time the tidal wave reaches the limit of the tidal current, the largest portion of the tide water which has entered the river has again returned to the sea. The remainder of the tidal water, together with the backed up river water, must then flow downstream. *A large amount of water entering the river results in a large amount of river water being backed up; this in turn causes a greater total quantity of water to flow at ebb and*

[1] Before the mixing, a stratification of fresh water over the sea water, the two flowing in opposite directions, takes place. This occurrence was observed by Plate, in connection with investigations on the Weser.

results in a stronger scouring force. This law forms the basis for all river-mouth corrections. It points out the necessity of bringing as much tidal water into the estuary as possible. The rules for doing this are discussed in following sections of this chapter.

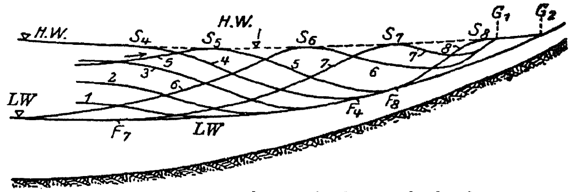


Fig. 100. Tidal waves in the mouth of a river

The tidal wave is a very flat wave. Its period is equal to the tide period of the sea. (In the North Sea this amounts to 12 hours, 25 minutes.)

Fig. 100 shows the upstream progress of a tidal wave as it occurs in a very long river such as the Amazon. The trace of the wave peak, S_4, S_5, . . ., S_7, is called the line of HW. The trace of the foot of the wave, F_7, F_4, F_8, etc., is the line of LW. If the wave crest requires more than the time of one period to move from the mouth to the limit of the flood current, several tidal waves will follow each other one after another. This occurs especially readily if the river has a large depth and flat slope, and the sea a large tidal range. (The Amazon and Yellow Rivers present examples of such conditions.)

The form of the tidal wave is dependent upon the shape of the river bed. If the river has a uniformly varying section and slope, the wave form will also be regular. If the bed possesses sudden widenings, the wave will sink at these points. At greatly narrowed regions the wave rises above normal. Both conditions result in a loss of kinetic energy and watei quantity. The same effects are incurred as a result of obstacles such as sand banks, groins, etc.

The velocity of the tidal current itself is different from the velocity of the tidal crest, although they both travel in the same direction. This is still more pronounced with regard to the ebb current, which is directed downstream while the ebb valley moves upstream, the directions being just opposite in this case. All waves which move up an inclined plane suffer retardation at the foot, that is, become steeper; this condition also occurs in rivers in connection with tidal waves. The tidal wave is steeper upstream than the ebb hill behind it. Hence, the period of rising is shorter than the ebb period. Figs. 101 and 102 show the interdependence of the tidal curve and tidal wave. The steeper the back tidal branch of the tide curve, the steeper must also be the front tidal slope of the wave. This upset of the wave is caused in a large measure by the impact of the discharging river water upon the foot of the wave.

If a river is of shallow depth in comparison to the tidal lift, or if it

possesses an especially strong current, the retardation may become so great that the tide actually breaks and rushes up the river as a steep wall of water or hydraulic bore. This wall-like wave, called hydraulic

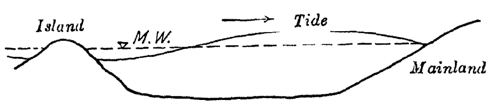


Fig. 101. Tidal wave

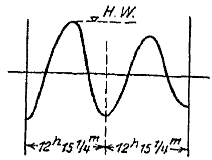


Fig. 102. Tide curve

Figs. 101 and 102. Relation between the tidal wave and tide curve

bore, reaches a height of 5 m. (16 ft.) in the Amazon River and races upstream at a velocity of 50 km. (31 mi.) an hour. The form of the wave varies in accordance with the diagrams shown in Fig. 108.

A good conception of the variation of tidal curves in rivers is given by tidal curves for the Elbe (Fig. 103). For example, the period of tide is only 4 hours and 46 minutes in Hamburg, while the ebb period is approximately 7.5 hours. Considering the instantaneous water stages, it will be observed from the figure that a tidal wave may reach from Cuxhaven to Hamburg. All tidal curves are referred to the same elevations. The velocity of movement of all points of the wave are approximately expressed by the formula,

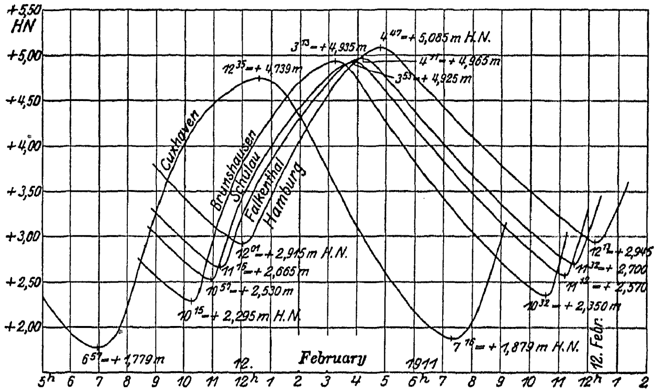


Fig. 103. Tide curves for the Elbe

$$V = \pm\sqrt{2g \cdot h} + v$$

in which $g = 9.81$ m. per sec., $h =$ the distance of the center of gravity of the profile from the top surface, and v denotes the usual velocity of flow of the river. For very wide rivers with steep banks and depth H for any arbitrary HW stage, the formula takes the practical form,

$$V = \pm\sqrt{g \cdot H} + v.$$

The equation is intended to hold true only if the wave height remains smaller than the river depth at LW. However, the formulas give values which are usually too large. According to Oeltjen and Dr. Reinecke, the formulas are not usable for accurate computations in irregular rivers, but may be used for computations on canals[1] (Suez Canal computed by de Thierry). In agreement with the formula, the velocity of the tidal crest is always greater than that of its base, so that the crest of every tidal wave partly overtakes the valley preceding it. This explains the shortening of the tidal period and lengthening of the ebb period (Fig. 103).

b. High-Water and Low-Water Lines

The high-water and low-water lines characterize the condition of a river. They indicate the following properties:

1. When mouths have the form of a trumpet, such as the Bristol Canal, the low-water line falls toward the point of the trumpet; on the other hand, the high-water line rises toward this point. The fall or rise of this mildly curved line may amount to over 10 m. (over 33 ft.), that is, the range may amount to more than 20 m. (66 ft.). The rise and fall is greater the more the cross-section narrows toward the upstream end. This sort of development is probably more frequently a property of sea bays than of rivers (Fig. 104).

2. In deep rivers with regular shore and bed conformation, the HW line rises somewhat at the beginning, extends practically horizontal through the larger part of the ebb and tide district, and then again rises rapidly to the junction with the LW line, that is, up to the tidal limit. The LW line falls from the sea toward the river and then gradually rises in a mild curve. The more regular the form of the river mouth, the more uniform will be the variation in fall of both lines (Fig. 105).

3. In rivers with rapidly changing width and depth, and containing forks, islands, etc., frequently the HW line rises and falls several times within the tidal stretch. It falls at points of abrupt widening and rises at points where the river channel abruptly narrows. However, a large amount of sea water must enter the river. In the case of such river mouths, the LW line lies higher and the HW line deeper; thus, the difference in elevation of the two lines is less than in the case of uniformly developed rivers (Fig. 106).

4. Only a small amount of tidewater enters the river if the mouth is not trumpet-shaped, and is narrow throughout. The tidal wave rapidly

[1] Oeltjen, Zentralbl. Bauverw. 1919, p. 137; Reinecke, *Berechnung der Tidewelle im Tideflusse*, 1921. Mittler & Sohn, Berlin. Further report by de Thierry to the International Navigation Congress, Philadelphia, 1912.

disappears, the HW line falls practically the entire distance from the sea toward the river. This condition occurs in most small rivers (Fig. 107).

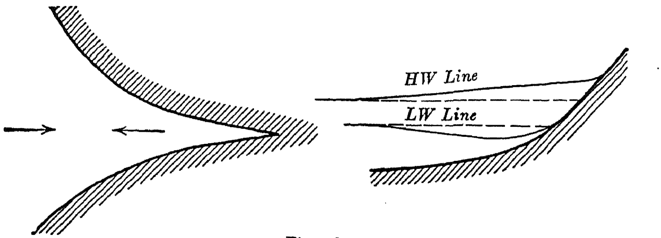


Fig. 104

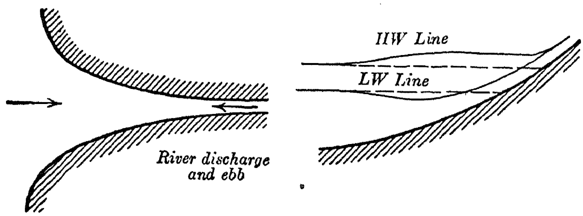


Fig. 105

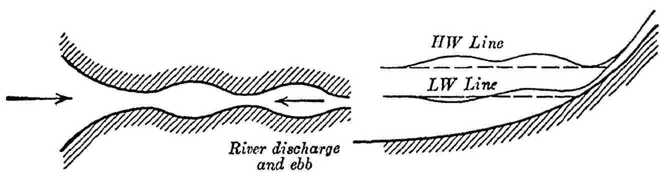


Fig. 106

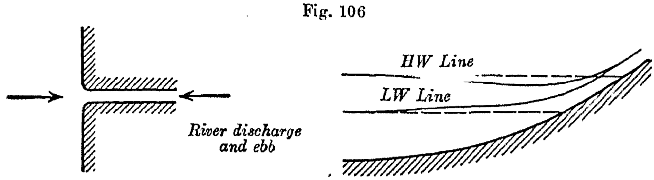


Fig. 107

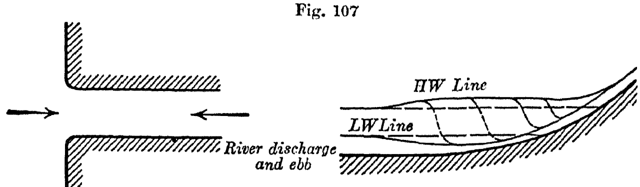


Fig. 108

→ = Flood
← = Ebb

Figs. 104–108. High-water and low-water lines in river mouths

5. In the case of broad, shallow rivers or deep rivers having a very strong current, the hydraulic bore occurs (Fig. 108).

Case two describes the condition which is to be striven for in the

correction of every river. The LW line will sink a maximum amount as a result of this type of correction, and the river will then develop the greatest possible difference between the elevations of the HW and LW lines. The tidal limit lies further upstream than in cases three to five. The river mouth, therefore, will possess the maximum length of deep water course at HW, and also the longest tide period.

c. Action of Variations in Water Stage upon the River and Sea

Up to this point the river flow and the tidal flow into the river were considered the same. However, such a condition is never maintained. Most frequently the flow from the river remains constant while the tidal height continually changes between spring tide and neap tide. The situation also arises in which the river discharge increases greatly within a few days while only small changes take place in the successive tidal rises. Such a combination of circumstances occurs, for example, if a sudden rise in the river comes at a time just before spring tide or neap tide. Various cases are considered in detail.

CASE 1. APPROXIMATELY EQUAL HIGH-WATER HEIGHTS IN THE SEA, INCREASING DISCHARGE FROM THE UPPER RIVER. When the river discharge increases, the resistance to tidal waves becomes greater. The energy of the tide will, therefore, be absorbed more rapidly and the wave will not go upstream as far. Tidal limit and tidal current limit lie further downstream than at the low stage of the upper river. Both the crest and base of the tidal wave are raised, the base more than the crest, so the tidal difference becomes smaller with increase in flow of the upper river, although the navigable depth increases. These changes in the tidal line are greatest in the upper tidal district and decrease toward the sea. However, for a long distance upstream from the mouth, the differences are not noticeable, particularly as far upstream as the quantity of sea water in the river remains several times greater than the quantity of river water.

CASE 2. VARYING TIDES IN THE SEA, UNIFORM FRESH WATER FLOW. A suddenly occurring increase in high-water of the sea (storm tide) pushes the tidal limit far upstream and raises both the HW and LW lines in the river, the HW more than the LW. As the time of spring tide is approached, the tide each time reaches further upstream and backs up successively larger quantities of river water. The HW line then rises over the entire stretch. The rising HW of the sea is accompanied by a sinking of LW. Accordingly, the LW line of the river must also lie deeper at the mouth for the lowest ebbs than for average tides. Further upstream, due to the backing up of the upper river water, a rise of the LW line occurs. The fall of the LW line at spring tide is

thereby increased, so that it cuts the LW line of neap tide at a point some distance upstream from the mouth. Fig. 119, showing longitudinal section of the Seine, indicates this point to lie at Vieux Port, a distance of 44 km. (27.3 mi.) from the mouth.

Thus, at spring tide, higher HW lines occur throughout the section of the river influenced by the tide; in the lower regions of the river, lower LW lines occur than at neap tide.

CASE 3. VARYING TIDES AND CHANGING DISCHARGE FROM UPSTREAM. The displacement of the tidal limit and the location of HW and LW lines is so variable for these conditions that no general rule can be set up.

Even storm tides of equal height may cause entirely different conditions in the river, depending upon whether strong, high tides have preceded or not. Invariably a high rate of discharge from the upper river under otherwise similar conditions, will flow with its surface at a lower elevation after correction than before, since the discharge condition is improved.

d. Determination of Water Quantities

Mouths with strong tides show the characteristic property that the water quantities in the mouth may be varied by intervention. The increase in quantity up to a favorable amount is one of the aims of correction measures, since this fosters further advantages for the improvement of the mouth. Increasing the water quantities which rush upstream as tides is, therefore, the first important step in the correction of a river mouth.

In order to determine the maximum tidal inflow which can be attained, and to find the corresponding channel form, the quantity of water moving back and forth in every important section of the river must be known. This knowledge forms the basis for drawing conclusions concerning the stream's behavior.

The quantity of water flowing through a section at any time can be determined only for a short time interval. This is done in the following manner: The water stage at several locations above the cross-section, as far as the tidal limit, is recorded by means of recording gages at the beginning and end of the tide for which computations are to be made. From these stage records the wave lines of the tide are superimposed on each other. For example, see Fig. 109 in which it is assumed

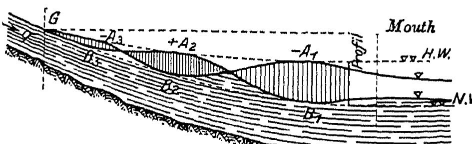


Fig. 109. Computation of water quantities

that two waves occur in the river. These waves have moved forward from the beginning to the end of the observation from A_1 to A_2 and from A_3 to G. At the beginning, all water in the river was below the line $A_1B_2A_3$; at the end of the period, the river still contained the water below the line $B_1A_2B_3$, which was less than before. The difference of the two quantities, therefore, is the amount discharged during the period of observation. To this must be added the quantity of water q which came in from above the stretch influenced by the tide. The amount of lowering in elevation of points above A_1 and A_3 are designated by s_1' s_1'' s_1''' and s_3' s_3'' s_3''', etc.; the rise of points above B_2, by h_2' h_2'', etc.; the mean horizontal water surface area belonging to each height, by $0_1'0_1''0_1'''$, $0_2'$ $0_2''$, etc. Furthermore, the constant inflow entering from above within the time interval considered is designated by q, then the quantity, Q, which was discharged through the section is

$$Q = -\Sigma s_1 \cdot 0_1 + \Sigma h_2 \cdot 0_2 - \Sigma s_3 \cdot 0_3 + q$$

or simplified,

$$Q = +\Sigma h \cdot 0 - \Sigma s \cdot 0 + q.$$

The average horizontal water surface area (values of 0_n) is determined simply as the arithmetical mean of the areas which can be measured from the curves taken at the beginning and end of the time period. The individual sums are the contents of the body which is limited above and below by the tidal water surfaces at the beginning and end observations, respectively. Thus in order to determine the quantity of flow past a section, it is necessary to have a record of the water stages for the entire district within the tidal range above the section. By beginning with cross-sections nearest the upstream end of the estuary, the quantity Q_n of each following section may be computed from the preceding Q_{n-1} by adding or subtracting $\Sigma h \cdot 0$ or $\Sigma s \cdot 0$, depending upon the relative position of the area.

The value of Q may be positive, negative, or equal to 0, depending upon the position of the section. Accordingly, water at the section will be either in ebb, flow, or subject to no movement at all.

A large number of gaging stations is necessary for accurate measurements. The water quantity is determined not only for a large number of cross-sections but also for several time periods between HW and LW. Furthermore, tidal changes which occur most frequently should particularly be chosen for measurement because the development of an estuary depends more particularly upon these occurrences. Extraordinary high or low tides should preferably be discarded because their effect is uncertain. With a large number of normal tide curves as a

basis, mean curves are drawn and the tidal wave curve determined therefrom. If average tidal curves of this nature for individual locations are grouped together, as has been done for the Weser in Figs. 110 and 111, they may be used to determine the shape of the tidal wave for any instant. By taking cognizance of the time of occurrence of the water stages indicated in the figure, the water stage at a definite hour

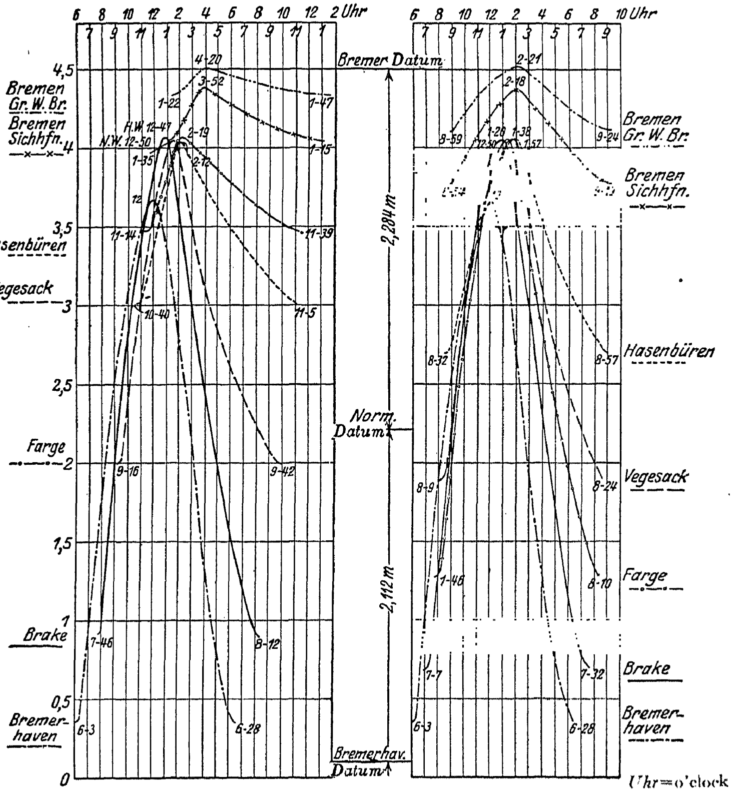


Fig. 110. Before the correction measures Fig. 111. After the correction measures

Figs. 110 and 111. Normal tide curves for the Weser River during low water in the upper course

for all localities along the estuary may be determined and plotted in a way which will show the shape of the tidal wave. Figs. 110 and 111 indicate the small influence of the tidal wave upon the stage in the upper district of the river and the relative importance of the tidal wave in the lower district.

This preliminary work is followed by the actual design of the correction works. The quantity of water which will flow upstream at rising tide after the correction is the actual unknown of the entire computa-

tion. It is dependent upon such assumed data as the form and size of various cross-sections, the elevation and the slope of the bed, etc. Considerable skill is required to determine these arbitrarily chosen data satisfactorily in such a manner that the assumed water stages actually occur, and thereby that the assumed quantities of water enter the mouth of the river. Definite sections can be readily chosen and evaluated, but resistances in the river may be so large that the water will not flow inward as far as assumed. If this turns out to be the case, it will be found by subsequent computations that the assumed sections are not entirely filled. They are, therefore, incorrectly chosen, because only the part of the section filled by water should be considered. Consequently, after choosing cross-sections, etc., it is necessary to evaluate the water quantity approximately at the outset. At first all hindrances existing before the correction, which have shown themselves to be unfavorable in the measured tidal-wave curve, should be considered removed. Improved tidal-wave curves may then be drawn assuming larger depths of bed and improved forms of the cross-sections.

The velocity of flow of the tide can be determined approximately by the aid of the velocity formula $V = \pm\mu\sqrt{2g \cdot h} + v$. From this formula the form of the tidal-wave and the ebb curves must be determined as accurately as possible by comparison with the curves available for the estuary before correction. The velocities and the newly determined wave curves are then used in computing the water quantities which must flow through the new sections. The computed water quantities indicate whether the newly assumed sections are too large or too small. This computation must be repeated several times, or at least until there is sufficient agreement in the results.[1] In order to judge whether the chosen cross-section is too large or too small, the rules for river regulation may be applied. If the velocity is too low, the bed will sand up; if it is too great, the tidal force will be consumed too rapidly because of increased frictional resistance. A loss in kinetic energy, $.5 \cdot m \cdot v^2$, is not only noticeable in the profile, but is an unrestorable loss to the entire upper tidal stretch, since a corresponding loss is incurred in the amount of tidewater entering the estuary.

[1] The correction of the lower Weser was computed in this manner by Ludwig Franzius. The investigations of Oeltjen and Dr. Reinecke showed that the older formula contains two large sources of error. New methods were developed which led to very close agreement between computations and measurements. They are given in the doctorate dissertation of Reinecke. One of the most interesting features of Reinecke's investigations is that it emphasizes the remarkable performance of an engineer who relied upon a highly trained sense of engineering judgment to design a layout such as the Weser correction successfully in spite of the fact that the formulas existing at that time were incorrect. (Plate, *Bremen und die Weser*, Werft und Reederei, 1923.)

e. The River Bed and the Means of Correction in the Lower Course

The aim of correction in the lower course as well as in the upper course is to obtain the greatest and most uniform deepening possible in the entire river bed. However, the means to attain this end is very different for the two stretches. In the upper river the same amount of water is utilized as flows in the uncorrected river. In the tidal district the quantity of water which enters the stream because of tide, should not only be carried upstream further than before correction, but should also be increased to several times the original quantity. The problem in the upper course is to narrow the wide and therefore the flat sections in order to cause a deepening of the channel; in the lower course, the problem usually consists of enlarging the section.

Ludwig Franzius[1] recognized the solution of the problem to be governed by the law: "The more freely the tidal wave can move past every point, the greater will be the quantity of water flowing upstream during tidal rise; this in turn will result in increased scouring force during ebb, or increased capacity to develop and maintain a spacious, especially a deeper, bed."

To this may further be noted: The actual value of tidal water lies less in its upstream current than in the return current at ebb. In fact, the inward flow resulting from a rising tide is useless because the scouring force is directed upstream. The detritus is even moved upstream and energy is thus consumed to no advantage. Hence the upstream conduction of the tide should be accomplished with the greatest possible saving of energy, so that the discharge of tidal and river water to the sea will have a maximum scouring capacity and transporting force. In order to attain this aim, all obstacles should be removed from the river as far as feasible, especially those which hinder and weaken the tidal wave.

The lower river does not require bends, as is the case in the upper course. They are even harmful. Theoretically, this is also true of mild bends, because the river length is increased by them. In practice, however, smooth, flat curves are not removed because of the cost which would be involved. It is important for the development of long straight stretches that the LW channel be accurately fixed and so situated that sedimentation cannot occur within its course. In order to guarantee this condition, the LW channel must be slender and must spread uniformly toward the mouth. Any sudden widening fosters irregular sedimentation. A perfectly straight channel is the ideal design for a corrected lower course.

Sharp bends force the water to change its direction abruptly and to

[1] *Handbuch der Ingenieurwissenschaften*, III, 3, 1901.

consume a portion of the kinetic energy of the tide. They are scoured on the concave side, become sanded up on the convex side, and thus the section becomes irregular; the water flows rapidly in one part of the channel and slowly in the other, the combination causing dissipation of energy. Sharp curves must be cut off or considerably straightened. The nearer the harbor lies to the tidal limit, the shallower it will be. At the tidal limit there is a rapid transition to the shallow depth of the upper course. If there is danger that the tidal wave will not flow by the harbor city at sufficient height, the large bends must also be eliminated in order to shorten the distance to the sea.

Short curves may be improved by excavating the projecting shores and filling in the hollow shore. Of course, the hollow shore may not be developed by means of groins; longitudinal guide dams must be constructed. The space in back of these dams is filled with earth or sand. In case of large radius bends, cut-offs are the only remedy; but in rivers with large tidal funnels, they may be used only in the upper tide district. In the lower district, the large width would require movement of enormous quantities of ground. The design of a cut-off requires consideration regarding the advisability of simply closing the old arm at the upper end and allowing it to stand as a basin (increase of tidal flow for the stretch downstream), or filling the old arm with earth (decrease in tidal flow compared to the first procedure). Filling will generally cheapen the correction, since frequently there is a lack of space for unloading excavated material. Keeping the space open may increase the effectiveness of the correction.

Forks: Islands and longitudinal sand bars generate forks. They enlarge the friction area and dissipate more kinetic energy than curves. If the arms at both sides of an island are of different length, water will flow more rapidly through the shorter and tidewater will enter the longer arm from the upstream end, causing destruction of another part of the tidal wave. All large forks should be removed; very small arms are usually unharmful and have little effect upon the main arm.

In closing river forks, the closing dam (Fig. 112) must invariably be constructed at the upper end of the arm, although it may become more expensive there. The dead arm then acts as a reservoir chamber, favorable to the lower tidal district. The entire arm is filled in some instances in order to save cost; this is recommended for the upper portion of the arm to an elevation

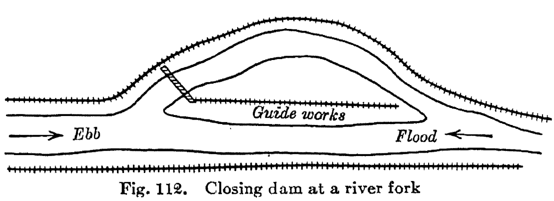


Fig. 112. Closing dam at a river fork

above HW because the closing dam will thereby be protected against the action of overflow at times of especially high tides. If two arms have the same width, the longer is cut off; if one arm is substantially wider, it should be developed, provided it is not significantly longer than the narrower arm. Individual circumstances determine the proper solution in each case and no general rule should be prescribed.

It is necessary to protect the closing dams against underscour. Wattlework should be laid if loose substrata are encountered. The dams must be built up as rapidly as possible so as to shorten the danger period for a break due to high-water. Since the arm remaining open must be widened by dredging, the latter operation should preferably be carried out simultaneously with the dam construction. In this way, the finished portion of the dam can be progressively protected by dumping dredged material on both sides.

Alternate narrowing and widening of the channel forces the high-water line to high or low elevations according to whether the breadth decreases or increases. In both cases energy is dissipated and the velocity of flow reduced. The size of the sections must increase uniformly downstream (Fig. 122).

Transverse bars consist of sand banks which extend across the entire width of the river and change their position very little. They are not only found in the mouth, but also upstream at the upper limit of tidal flow because in this vicinity there is a regular recurrence of backwater without strengthened scour action after the tide recedes. This part of the river particularly requires dredging operations. The most certain means of ridding the river of bars at the mouth is the combined action of a strengthened tidal current and dredging.

If the tidal rise is great enough, the bars do not obstruct navigation but require deep vessels to cross only at the higher water stages. Consequently, steamships are often forced to wait in front of the bars for practically an entire tidal period in case they reach the bar just after high water. The elimination of outer bars by means of structures is usually not practicable. In this case help can be obtained only by dredging.

The various fundamental structures for river control, such as cover works, guide dams, etc., are also used in improving the river mouth. However, the structures usually serve different purposes in the two instances.

The enclosure of the LW and MW channels may not extend above the adjòining HW channel because at ebb the tidewater at the sides must be free to flow unhindered into the LW channel. Actually, for well-corrected rivers at rising tide, water flows from the center toward the

sides; at ebb, from the sides toward the center. Low guide dams are cheaper to construct and maintain than those extending above LW. Fig. 113 indicates the design for a guide dam for the lower channel of the Weser. High-water guide dams are made substantially stronger. One for the Elbe below Hamburg is shown diagrammatically in Fig. 114. The

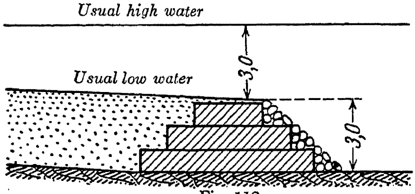


Fig. 113.
Guide dams of the Weser for low-water regulation

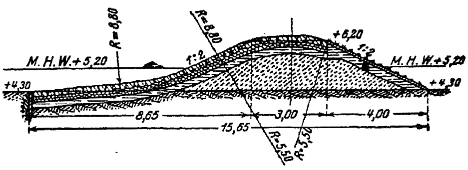


Fig. 114. High-water guide dams of the Elbe

structure consists of a sand core with clay cover; the river side is protected by paving, the bank side by sod. Guide dams are suitably filled in at the back with dredged material. They are prolonged toward the upper tide district, gradually being raised to the elevation of the higher parallel works, and are frequently tied into the shore with transverse embankments.

The HW channel does not require as careful development as the LW channel. It must be as wide as practicable but without too great irregularity. If economically practicable, this channel should be made as uniform as possible by fill and excavation. A very exact equalization is not necessary, however, because the current is small at the edges of a wide bed. As land is frequently desired for hydraulic fill, it can in many cases be reclaimed, while at the same time the HW channel will be made more uniform.

In all events, special emphasis must be laid upon obtaining as little friction in the channel as possible. The roughness of the bed may be natural (projecting rocks and crags) or created artificially (installation of groins) through lack of understanding. The installation of numerous groins in the tidal district is especially harmful because of the energy-consuming influence of these structures. They can cause a tidal wave, which would otherwise be of sufficient strength, to be destroyed within a short time. The installation of individual groins on the other hand will usually not have great influence upon the quantity of tide water and may be particularly advantageous locally. The tidal district requires much more dredging during and after the correction than the upper stretch. Dredging is especially required in the two cases discussed below.

First, dredging is required when the current, during the installation of regulation works, is not capable of loosening the ground of its own accord, and in order to accelerate the work of regulation. For example,

a large sand bank may close the entrance to tides. If this bank is quickly dredged out, the tide will carry off sand from further upstream while the work of correction is still progressing.

Secondly, dredging is required after the construction has been completed, at the upstream edge of the tidal district as well as in the central portion thereof. The upper limit of the tidal district readily becomes sanded up. A deep channel can be maintained at this point only through continual dredging, as the current strength alone is not sufficient to move the material. In the middle tidal district, deposition may be caused by especially high upstream flow or by storm tides. These deposits will gradually disappear after normal conditions again prevail. But heavy river traffic should not undergo long periods of disturbance. In its interest, the rapid removal of deposits through dredging is necessary. Provision should be made simultaneously with the design of the correction for a sufficient number of dredges and flat-bottomed lighters. The use of steam-driven, flat-bottomed lighters is particularly recommended because they decrease the cost of transportation of dredged material.

With reference to transportation requirements, the following rule for correction is applicable. Where deep draft is required, ships coming from sea at HW ride the HW wave, so to speak, and move approximately at the velocity of the wave crest. Ships traveling toward the sea, travel against the wave and consequently have worse navigation conditions. The correction should be so developed, therefore, that the outgoing ships can reach the deep water within the period of one tide. (See description of the Weser correction.)

f. Examples of the Correction of Lower Courses of Tidal Rivers

The following descriptions of several corrections concerns the various methods of procedure used to obtain the desired result. In no realm of engineering art, however, is it more difficult to generalize on the basis of individual cases.

Noteworthy examples of correction include the Clyde, Tyne, Tees, Mersey, Seine, Loire, Garonne, New Maas, Schelde, Ems, Weser, and Elbe, and further the Hugli (western main arm of the Ganges). The Clyde, Seine, and Weser are treated in the following discussion.

1. The Clyde

After a course of 158 km. (98 mi.) the Clyde mouths into a deep, wide bay, the Firth of Clyde, which is completely free of sand banks. The river drains an area of over 2,400 sq. km. (930 sq. mi.). Its mouth district is 34 km. (21 mi.) long; its upper tidal district, some 31 km. (19 mi.) long.

The most important city on the Clyde is Glasgow, the industry and commerce of this city making correction necessary. As late as the year 1773, the river (Figs. 115 and 116) possessed a depth of less than 1.5 m.

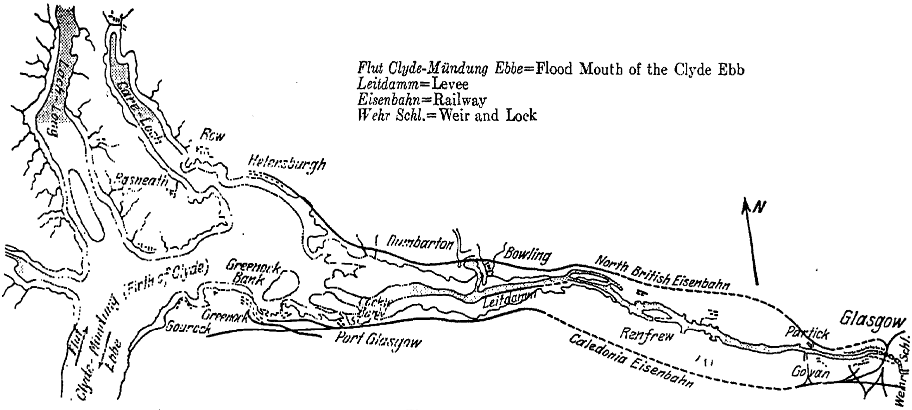


Fig. 115. Plan

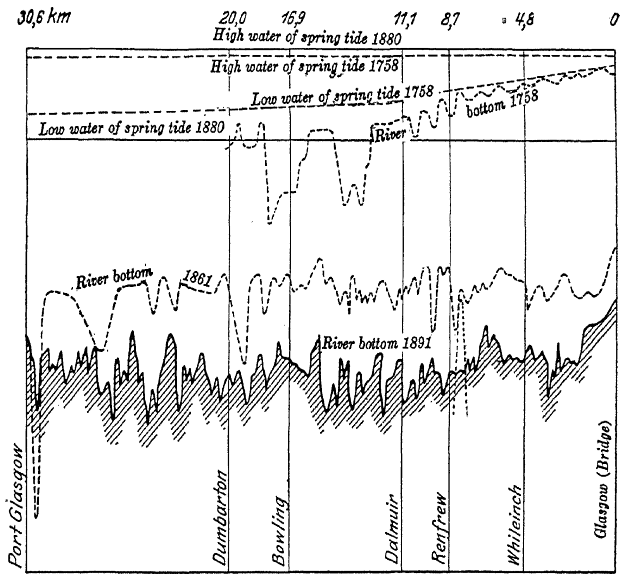


Fig. 116. Profile

Figs. 115 and 116. Mouth of the Clyde River

(4.9 ft.) at LW 19 km. (11.7 mi.) downstream from Glasgow. Work was begun at this time on the first substantial correction, but operations were limited to a stretch of 20 km. (12.4 mi.) below Glasgow. This stretch of

the river was gradually narrowed by the installation of groins, from 212 m. (695 ft.) at Dumbartin to 55 m. (180 ft.) below Glasgow. Probably inasmuch as the groin fields extended only over this short stretch [20 km. (12.4 mi.)] and did not weaken the tidal wave up to Dumbartin so that there was a strong scouring effect, success was attained in spite of the incorrect use of groin construction. The shallow water at Dumbartin was deepened to 4.27 m. (14.01 ft.) and the tidal variation at Glasgow was brought to over 2 m. (over 6 ft.).

In the year 1836 a design for the entire tidal district was prepared and put in effect. Most of the groins were removed and the new shore line was fixed by guide dams. The river width between guide dams below Glasgow now amounts to 113 m. (371 ft.) and 305 m. (1,000 ft.) at Dumbartin.

The effect of the river correction is supplemented by a large amount of dredging. Fig. 116 indicates the extent to which deepening succeeded. Maintenance of this depth would not have been possible without dredging. On an average, over .5 million cu. m. (over .65 million cu. yds.) of sediment must be dredged annually. Part of the dredged material is used to fill low districts which become valuable for factory sites.

The value of this course of procedure should not be underrated,[1] because, although a great amount of dredging is required, a much greater quantity of sand is removed by the current itself. The principal success is the valuable increase of tidal variation within the river. The tidal change at Glasgow, mainly as a result of sinking of the ebb level 2.59 m. (8.49 ft.), has increased 2.67 m. (8.75 ft.), so that the total change now amounts to 3.43 m. (11.25 ft.). In 1912 the navigable depth at LW varied between 5.2 and 6.1 m. (17.05 and 20 ft.).

The cost of the actual correction works was 43.4 million marks (10.3 million dollars). The apparently high cost is probably explainable by the faulty method of correction used in the beginning. In its present condition the project may be designated as a success.

2. The Seine

The Seine is 770 km. (478 mi.) long and drains an area of 77,800 sq. m. (30,000 sq. mi.). The mouth district is 20 km. (12.4 mi.) long and the upper tidal district about 150 km. (about 93 mi.) long, reaching from Berville to Le Havre. The tidal limit lies 19 km. (11.7 mi.) above Rouen. The tide interval is 7.5 m. (24.6 ft.) at Le Havre and 2.2 m. (7.2 ft.) at Rouen for spring tide, and 2.30 m. (7.54 ft.) at Le Havre and 1.00 m. (3.28 ft.) at Rouen at neap tide. Fig. 117 shows tide curves beginning at the sea (Barfleur) over a distance of 100 km. (62 mi.) to Le Havre. The

[1] *Engineering*, Vol. 1, 1912, p. 29.

backing up of the tidal wave is readily recognized. Fig. 119 shows the course of the tidal curve in the Seine up to Rouen.

The Seine discharges a maximum of 2,500 cu. m. (88,500 cu. ft.) per sec. and a minimum of 200 cu. m. (7,080 cu. ft.) per sec., an average of about 485 cu. m. (17,169 cu. ft.) per sec., and transports very little sediment. Before the correction (1848) the navigable depth between Rouen and la Mailleray was regular and sufficiently deep. Further downstream it was very irregular. The many sand banks found in this district continually changed their position. The bed had a width of 1,000 m. (3,280 ft.) at Tonkerville and widened to 10,000 m. (32,800 ft.) at Honfleur. Below Quilleboeuf the depth was only 4.3 m. (14.1 ft.) at spring tide and 1.75 m. (5.74 ft.) at low tide. A dangerous bore developed above this point.

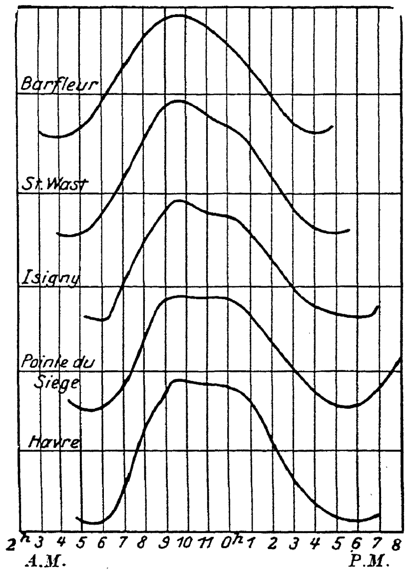


Fig. 117. Tide curves in the Seine Bay

The first correction was begun in 1848 and completed in 1866. It consisted mainly of guide dams constructed between a point above Caudebec and the town of Berville which lies some distance downstream, and dredging operations between la Mailleray and Berville. The distance between the guide dams from Caudebec to Quilleboeuf was 300 m. (984 ft.) and from there to Berville increased to 530 m. (1,738 ft.). The dams were to reach above MHW at spring tide and to carry off the entire HW flow.

This work has materially improved navigation of the river. The earlier shallow places have disappeared, while at individual locations the bed has sunk 3 to 4 m. (10 to 13 ft.). The water depths have increased to the extent that at spring tide ships of 7 m. (23 ft.) draft, at low tide ships of 5.6 m. (18.4 ft.) draft can go upstream as far as Rouen in one tide period (Fig. 119).

The LW line has lowered very little as a result of the correction, at spring tide being about 2 m. (about 7 ft.) at Quilleboeuf, 1.5 m. (4.9 ft.) at Caudebec, and .8 m. (2.6 ft.) at Rouen. This small lowering of the LW line corresponds also to only a small increase in tide fluctuation which at spring tide is 7.5 m. (24.6 ft.) at Le Havre, only 3 m. (10 ft.) in Caudebec, and .5 m. (1.6 ft.) at Rouen. The HW line does not rise but has prac-

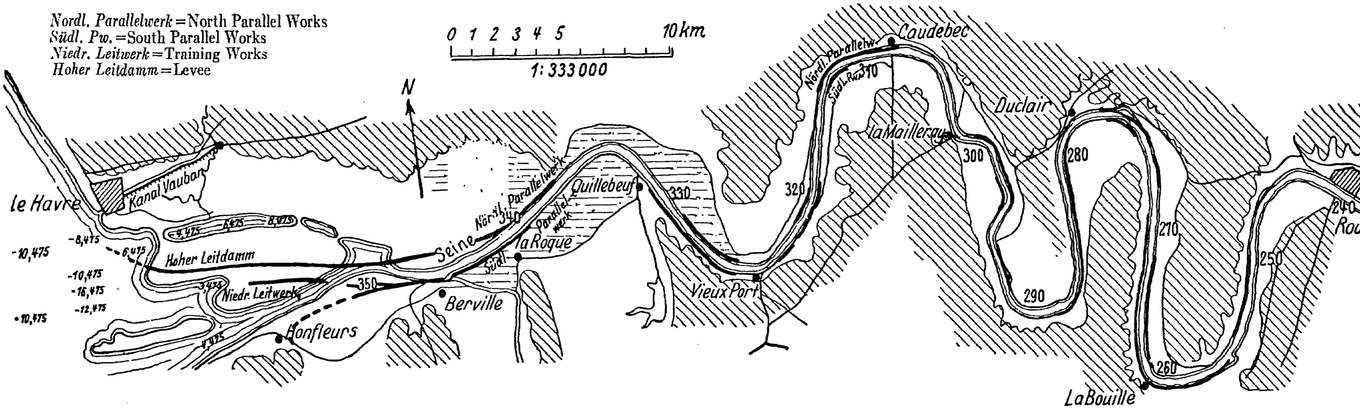


Fig. 118. Plan

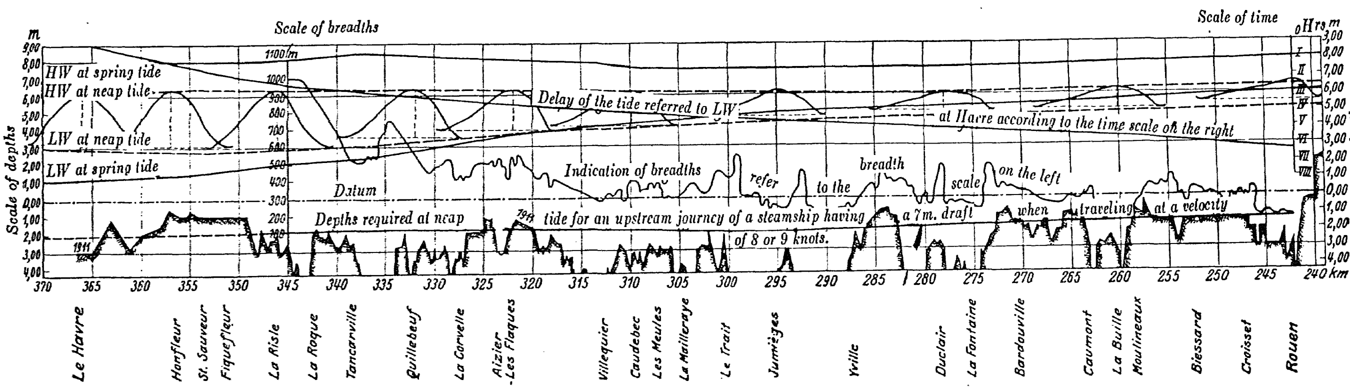


Fig. 119. Profile

Figs. 118 and 119. Tidal district of the Seine between Le Havre and Rouen

tically a horizontal course, showing a continuous, rapid diminution of the kinetic energy of the tidal wave.

This dissipation of kinetic energy is partly due to the steep slope of the LW line, and results in a strong, low-water current and hindrance to the high-water wave. It is also partly due to the small HW cross-section which is narrowed down too much by the HW levees. The high-water wave travels as it would in a canal; the energy taken from the wave results in downstream flow and consequent diminution of the wave height. If the section decreased in width uniformly toward the upstream direction, the weakening tidal wave would continually enter a narrower section and lose a lesser amount in height.

In spite of all the disadvantages of this type of correction, success was attained in the river itself but not in its mouth district.

Because of the extreme amount of contraction of the high-water channel, although there was a lowering of the LW line, the inflow of tidewater was diminished to the extent that a permanent change for the worse has taken place at the mouth. According to computations made by several hydraulic engineers, the decrease in quantity of tidewater entering the mouth and flowing past Le Havre, because of deposition which has occurred, amounts to 100 million cu. m. (3,500 million cu. ft.), while the diminution of space for tidewater in the entire tidal district amounts to 370 million cu. m. (13,000 million cu. ft.). This reduction corresponds to a substantial lessening of scouring action and greatly increased sedimentation in the form of sand banks and bars in the mouth district.

The amount of deposition in the mouth became so large that safe navigation was rendered impossible; several years later the entrance to the harbor of Le Havre was endangered. A new entrance had to be created by a special canal which branched off from the harbor of Le Havre and entered the Seine at Tonkersville. The canal is closed at both ends by locks. To assure entrance from Le Havre itself, a new entrance had to be made by the construction of long, expensive moles. The entrance is open at the west and lies outside of the district subject to deposition.

A new design for the correction of the Seine was made and carried out since 1895. Downstream from Quilleboeuf, a specially widened LW channel with an adjacent high-water channel has been constructed in order to again attain greater scouring effect of the tidewater. The position of the correction structures, as of the year 1911, is shown in Fig. 118. The same figure indicates the layout for the development of the earlier narrow mouth into a trumpet shape. The superimposed width line (Fig. 119) shows decrease in width at La Risle from 1,100 m. (3,608 ft.) to

about 170 m. (about 538 ft.) at Rouen. Fig. 120 shows the step-like development of the guide works which make use of the deposition and back scour behind the embankment, the superimposed structure each time being built upon the natural deposit. A vigorous dredging program has been carried on in the river since 1905, and in the mouth since 1917.

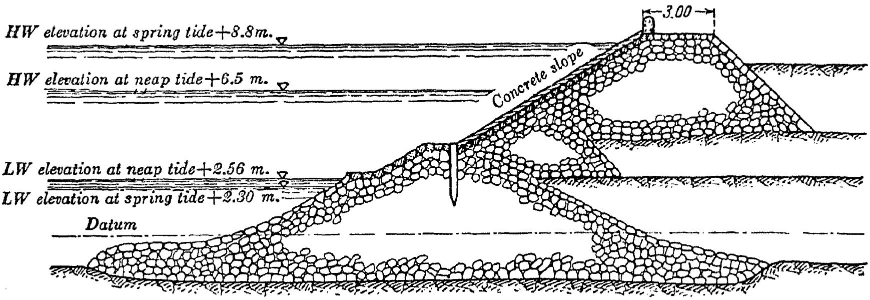


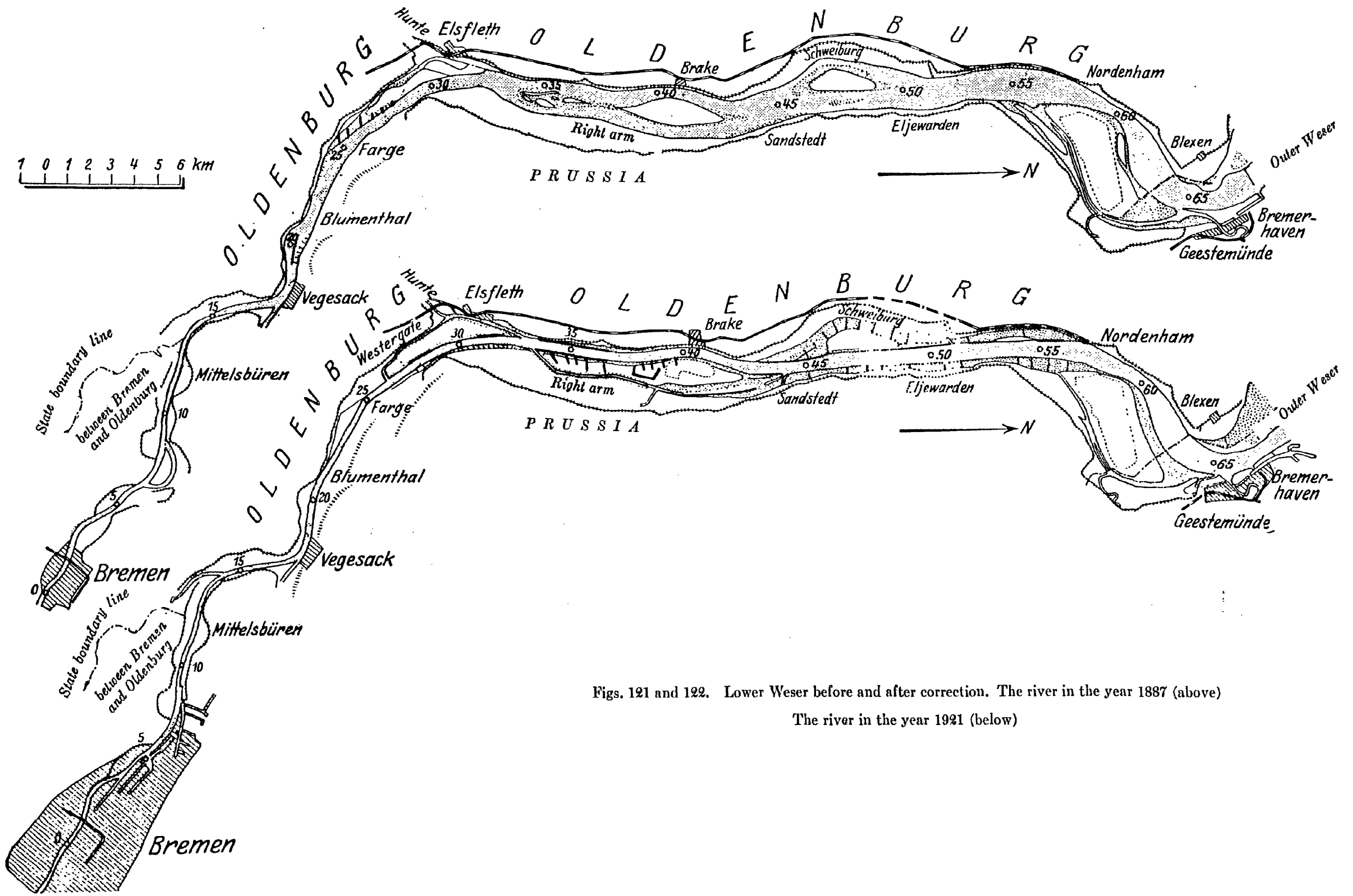
Fig. 120. Bank protection in the mouth district of the Seine

3. The Weser

The Weser rises in the Thüringer forest, is called Werra (the same word as Weser) over the first part of its course, and joins the Fulda at Hanover-Münden. It has a total length from the Werra source to Bremerhaven of 704 km. (437 mi.). The length of its upper tide district from Hemelingen weir to Bremerhaven is 73 km. (45 mi.) with the tidal limit at the Bremen weir. The mouth district may be assumed to have a total length of 55 km. (34 mi.), which includes a length of 45 km. (28 mi.) to the Red-Sand light tower.

The maximum high-water quantity arising inland amounts to 4,600 cu. m. (162,840 cu. ft.) per sec. at Bremen. The summer middle HW is 540 cu. m. (19,116 cu. ft.) per sec., the MW discharge 290 cu. m. (10,200 cu. ft.) per sec., and the LW discharge 100 cu. m. (3,540 cu. ft.) per sec. The quantity of water which passes Bremerhaven at a stage 1.5 m. (4.9 ft.) over the usual high water is 12,000 cu. m. (424,800 cu. ft.) per sec.; the average quantity of tidewater which then moves in the mouth amounts to over 100,000 cu. m. (3,540,000 cu. ft.) per sec.

Even the highest water from upstream has no effect upon the bed as far upstream as Brake, which is 27 km. (16.76 mi.), or about 2/5 of the length of tide district upstream from the mouth. The MW stood at elevation .73 m. (2.39 ft.) before the correction, the usual stage at .5 m. (1.6 ft.) on the Bremen gage. The average tide interval at Bremerhaven amounted to 3.3 m. (10.8 ft.). The condition of the river before the correction was as follows: In the 40 km. (25 mi.) stretch between Bremerhaven and Vegesack, the river was in a wild condition with the exception



Figs. 121 and 122. Lower Weser before and after correction. The river in the year 1887 (above)

The river in the year 1921 (below)

TABLE NO. 1

Duration of Elevations Corresponding to Mean Tides; Rates of Progression and Velocities Before and After Correction

A. Before Correction

I. Normal Tidal Wave during average flow condition from upstream = 0.73 m. on the "Grosse Weserbrücke" (Weser Bridge) at Bremen

Designation of the Section According to Locality	Distance to the Section		Height on the Gage: I. Low Water		Height on the Gage: II. High Water		Height of Tide		Duration: Tide		Duration: Ebb		Period of Progression: I. Low Water		Period of Progression: II. High Water		Velocity of Progression: I. Low Water		Velocity of Progression: II. High Water	
	km.	mi.	m.	ft.	m.	ft.	m.	ft.	hrs.	min.	hrs.	min.	hrs.	min.	hrs.	min.	m. per sec.	ft. per sec.	m. per sec.	ft. per sec.
Bremerhaven			0.26	0.85	3.56	11.68	3.30	10.83	5	57	6	28								
	26.93	16.72											1	42	—	47	4.30	14.11	9.50	31.17
Brake			0.97	3.18	4.11	13.48	3.14	10.30	5	0	7	25								
	14.80	9.19											1	30	—	46	2.74	8.99	5.30	17.39
Farge			1.07	3.51	3.02	9.91	1.95	6.39	4	17	8	8								
	8.67	5.38											1	42	—	41	1.40	4.59	3.52	11.55
Vegesack			1.02	3.35	1.93	6.33	0.91	2.99	3	16	9	9								
	8.54	5.30											—	56	—	35	2.54	8.33	4.07	13.35
Hasenbüren			0.84	2.76	1.10	3.61	0.26	0.85	2	55	9	30								
	8.46	5.25											1	39	1	34	1.42	4.66	1.50	4.93
Bremen, Sicherheitshafen			0.56	1.84	0.66	2.17	0.10	0.33	2	50	9	35								
	1.63	1.01																		
Bremen, "Grosse Weserbrücke"*			0.73	2.40	0.73	2.40	0	0												
Between Bremerhaven† and Bremen, "Grosse Weserbrücke"	69.03	42.86											7	29	4	23	2.50	8.20	4.27	14.01

* Tidal Limit. † Mean velocity of progression considering the length of individual stretches.

B. After Correction

II. Normal Tidal Wave during average flow condition from upstream = 0.73 m. on the "Grosse Weserbrücke" (Weser Bridge) at Bremen

Designation of the Section According to Locality	Distance to the Section		Height on the Gage				Height of Tide		Duration				Period of Progression				Velocity of Progression			
			I. Low Water		II. High Water				Tide		Ebb		I. Low Water		II. High Water		I. Low Water		II. High Water	
	km.	mi.	m.	ft.	m.	ft.	m.	ft.	hrs.	min.	hrs.	min.	hrs.	min.	hrs.	min.	m. per sec.	ft. per sec.	m. per sec.	ft. per sec.
Bremerhaven			0.26	0.85	3.56	11.68	3.30	10.83	5	57	6	28								
	26.96	16.74											1	4	—	50	7.0	22.97	9.0	29.5
Brake			0.76	2.49	4.11	13.48	3.35	10.99	5	43	6	42								
	14.80	9.19											—	38	—	30	6.5	21.33	8.2	26.9
Farge			0.28	0.92	3.07	10.07	2.79	9.15	5	35	6	50								
	8.67	5.38											1	3	—	27	2.3	7.5	6.1	20.01
Vegesack			0.01	0.03	1.93	6.33	1.94	6.37	4	59	7	26								
	8.54	5.30											—	40	—	26	3.6	11.8	5.5	18.05
Hasenbüren			0.02	0.07	1.10	3.61	1.12	3.67	4	45	7	40								
	7.41	4.60											1	—	—	57	2.4	7.9	2.5	8.2
Bremen, Sicherheitshafen			0.34	1.12	0.66	2.17	0.32	1.05	4	42	7	43								
	1.63	1.01											—	19	—	5	1.4	4.6	5.03	16.50
Bremen "Grosse Weserbrücke"			0.60	1.97	0.80	2.62	0.20	0.66	4	28	7	57								
	6.82	4.24											—	—	—	—	—	—	—	—
Habenhausen			0.60	1.97	0.60	1.97	0	0	—	—	—	—								
Between Bremerhaven† and Bremen, "Grosse Weserbrücke"	67.98	42.22											4	44	2	55	4.0	13.1	6.6	21.7

* Tidal Limit. † Mean velocity of progression considering the length of individual stretches.

of a stretch 5 km. (3 mi.) long. The wild stretch of river possessed a very irregular bed, and about half of the stretch was divided by islands and contained sand banks which laid dry during ebb. The course from Elsfleth to Farge and from Vegesack to Bremen had already been corrected.

A conception of the tide conditions may be obtained by comparing Fig. 121 with Fig. 122 and the table of average tides (Table 1). It will be noted that below Brake the tide conditions, and to a certain extent the relative depths, were favorable and the velocities high (last column of Table 1, A and B). Above Brake, the bed rose abruptly, the same being true of the LW line (Fig. 125). The navigable depth was, therefore, very small. The velocity of the water and the period of tide were reduced abruptly at this point. These unfavorable circumstances were caused by numerous forks in the river and irregularities in the bed above Brake, resulting in the rapid destruction of the kinetic energy of the tidal wave.

The tidal wave suffered the greatest loss between Brake and Vegesack, where the LW line was steepest. On the corrected stretch to Bremen, the slope of the LW line between these two points was reduced.

The worst part of the course from Brake to Vegesack was the stretch between Brake and Elsfleth. Upstream from Elsfleth to Vegesack, the river had been corrected by the construction of groins which narrowed the stream to the extent that the velocity was diminished enough to allow sedimentation. Below Elsfleth, the velocity was very low because of extreme widening. The sediment which would otherwise have covered the bed uniformly between Vegesack and Brake was transported and deposited below Elsfleth where a bar had formed.

With the lower river in the condition described, it was desired to improve the course between Bremerhaven and Bremen in a manner such that a more favorable condition would be maintained by the natural current with the aid of little artificial maintenance. No unfavorable influences were to be suffered in the mouth district as a result of work further upstream.

In order to satisfy this requirement, especially with reference to the mouth district, work was to be centered on increasing the quantity of tidewater entering the river and lengthening the tide district as much as possible.

All larger river forks were removed. As far as necessary, the arms which were cut off were retained open at the lower end and used as collecting basins. The downstream portion had to be widened to provide enough water for the collecting basins without damaging the upper part of the river. The channel was so formed that its section became uniformly smaller from Bremerhaven to Bremen (Figs. 124a and 124b). A double cross-section was used up to Vegesack and from there to Bremen

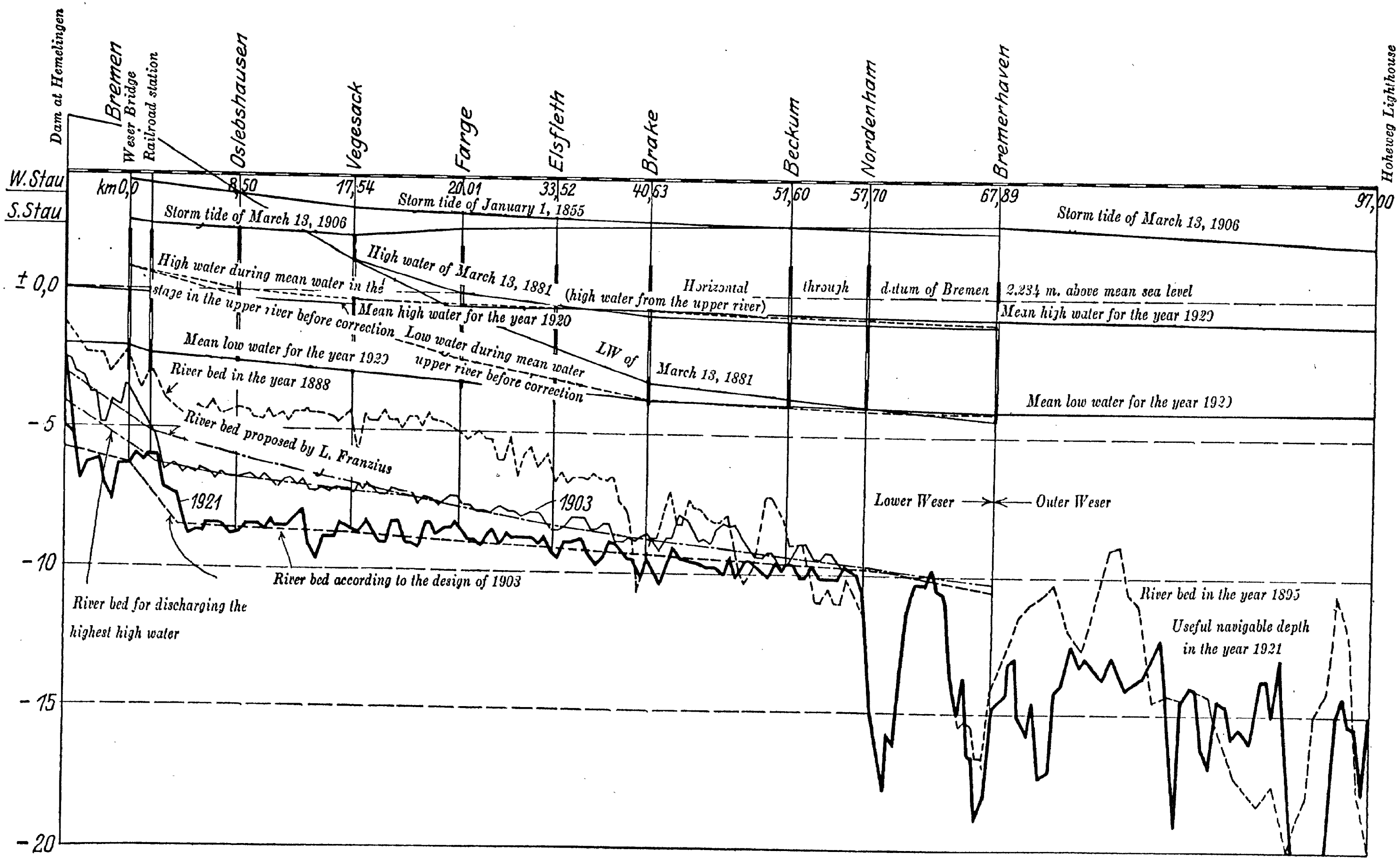
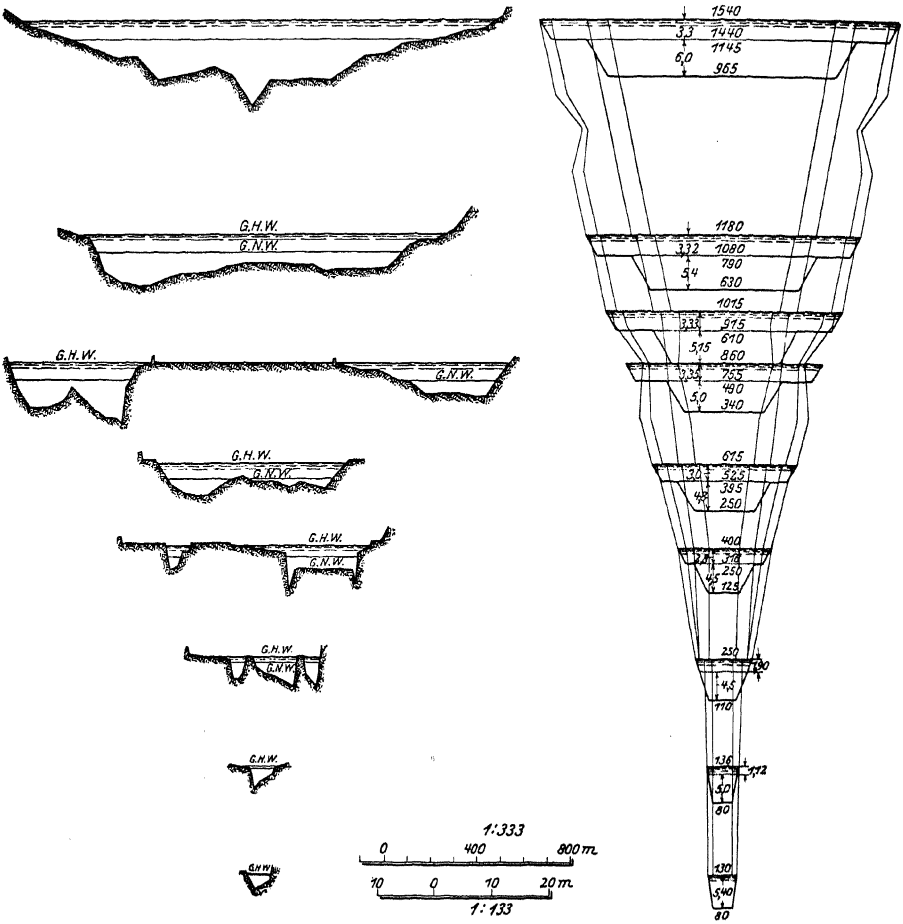


Fig. 123. Profile of the lower Weser. Design of 1903

a single section was constructed. The double cross-section consisted of a narrow LW channel and a broad LW channel which were arranged symmetrically part of the distance, the LW channel being contained in the center of the HW channel.



Figs. 124 a and b. Cross-section of the Weser River before and after the first correction

G.H.W. = Usual High Water *G.N.W.* = Usual Low Water

The depth of the LW channel was designed to make use of the existing depths as far as practicable to reduce the earthwork to a minimum. The HW channel was arranged to avoid the loss of valuable lands.

Preliminary computations of the water quantities to be expected and the corresponding necessary cross-section sizes made it possible to fix the

guide dams at the outset so that subsequent relocation did not become necessary.

Groins were not used. All guide dams were constructed of brush work, for the most part, in the form of wattlework weighted down by rock (Fig. 113). Two procedures were followed in the construction of the guide dams: Either the guide dam was completed immediately, or it was built in stages by constructing superimposed sills behind which sedimentation was allowed to take place during the time interval between the construction of each sill. In the first case, cross-ties were fully constructed and then the guide dams were built joining the ends of the ties. In the second case, the sills were completed only after the sedimentation had taken place. Where protection against floating ice or attack from water was necessary, the construction used consisted of heavy stone packing between woven fences.

A total of 55 million cu. m. (76 million cu. yd.) of earth had to be moved, 8.6 million cu. m. (11.3 million cu. yd.) were above low-water. Some 24 million cu. m. (31.5 million cu. yd.) were to be carried away by the current and about 31 million cu. m. (40.7 million cu. yd.) through dredging. The sediment carried by the current was not to be deposited at any point in the river but in the mouth district. The latter, with an area of 53,000 hectors (131,000 acres), is so large that this deposit is negligible compared to the sediment brought in by the sea.

The first correction came up completely to the highest expectations and in some respects materially surpassed them. The work may be considered a pattern solution of this type of engineering problem. In 1886 a navigable depth of only 2.75 m. (9.02 ft.) was available; eight years later (1894) the desired minimum depth of 5 m. (16 ft.) along the whole stretch was attained. The discharge of high-water from the upper Weser through the city of Bremen was better than formerly. The floods which used to occur occasionally in certain parts of the city were eliminated. The storm tide did not extend further upstream than formerly; neither did the salt water limit move further upstream. Both would have been harmful to the marshes bordering on the Weser. The velocity of the current, as may be seen from the Table, increased practically throughout the course influenced by tides and the amount of water entering all parts of the estuary increased greatly.

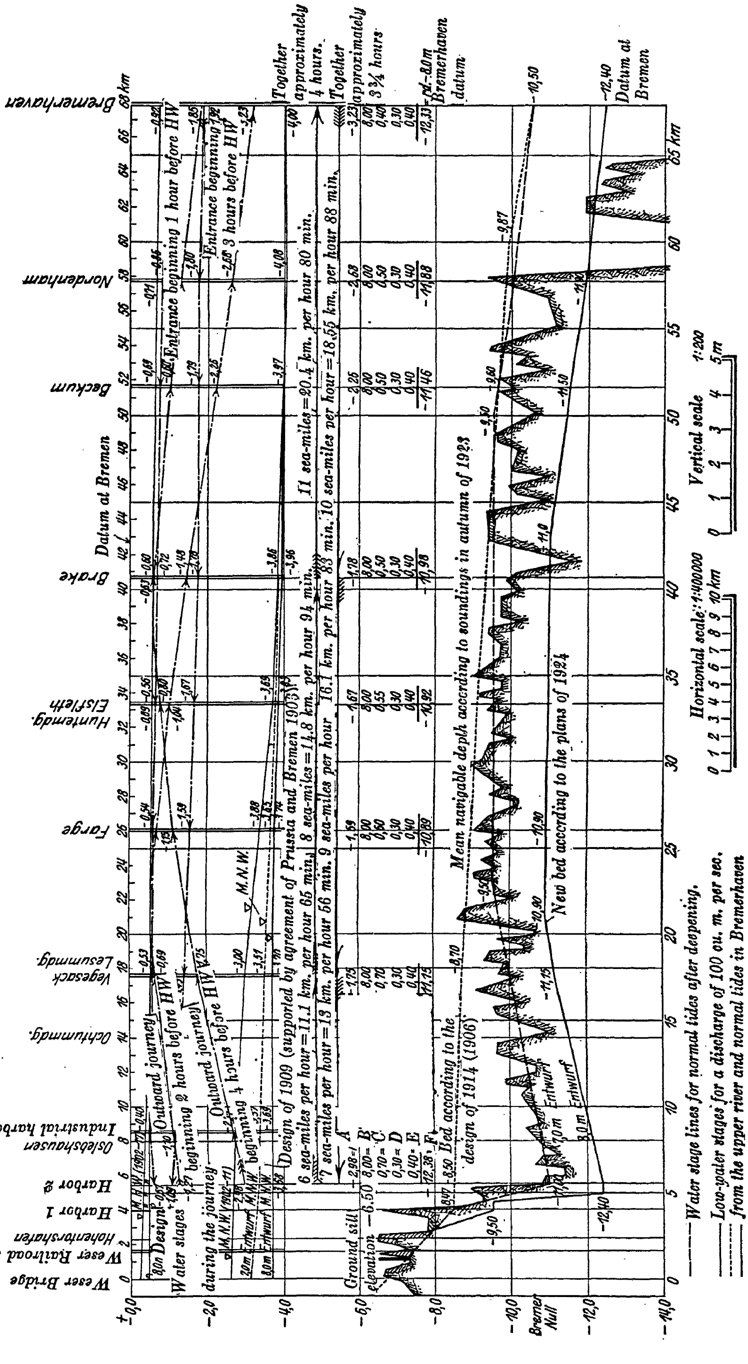
The location of the LW channel did not change noticeably. This was a result of the divided profile. The use of tide collectors, in the form of tributary arms and cut-off forks, acted favorably.

A particular effect of the Weser correction, according to Plate in an article on "Bremen u. die Weser" in the magazine *Werft Reederei u. Hafen*, 1923, translated, reads as follows: "The most noticeable occur-

rence of the development of the lower Weser was the change in tidal rise above Brake and especially at the city of Bremen. The change developed to a much higher degree than had been previously expected. At the gage on the large Weser bridge in Bremen, not only a large drop of LW amounting to 1.5 m. (4.9 ft.) has taken place but also a lowering of the HW, although to a much smaller degree, namely .58 m. (1.90 ft.). The tidal rise increased from .19 m. (.62 ft.) to 1.11 m. (3.64 ft.), while an increase of only .4 m. (1.3 ft.) with no lowering of the HW stage was expected. This large drop in water elevation was not harmful to navigation in the course of correction, because as a result of good discharge conditions at all times the position of the bed above Vegesack became 1.10 m. (3.60 ft.) deeper than was assumed in the original design. The extraordinarily favorable effect of the lower Weser correction upon the discharge of large quantities of water from further upstream was evident by a marked drop of the water stages. There was also less danger from storm tides occurring at the time of high water from the upper river, because the basin had greater storage capacity. The tide district extended some 10 km. (6 mi.) further upstream than before correction. Sinking of the normal groundwater level and also the lowering of the mud-laden spring HW stage acted disadvantageously on the Prussian marshes above Bremen. As previously mentioned,[1] this disadvantage had to be overcome in the Weser by constructing a backwater step above the city. On large, low areas where drainage conditions were improved, the lowering of water stages of the Weser (which also extended to tributary rivers) proved very favorable, and there is no doubt that the advantages to agriculture of the lower Weser correction were much greater than the disadvantageous effects."

Ludwig Franzius developed the plans for the freight ships of that time, about 1886, of 1,000 to 2,000 tons net capacity and 5 m. (16 ft.) draft. By 1900 the size of ships had been increased up to 2,000 to 3,000 tons net capacity and 7 m. (23 ft.) draft. The lower Weser, therefore, had to be further developed. A plan drawn up in 1903 was held in abeyance until 1913 because of difficulties with the other bordering states. At this time a development of the lower Weser for ships of 7 m. (23 ft.) draft was undertaken and practically completed by 1916, the World War notwithstanding. The sea ships constructed in the years 1918 to 1920 included 220 ships of 4,000 to 5,000 Br. R. T. capacity, 1,556 ships from 5,000 to 8,000 Br. R. T., and 110 ships from 8,000 to 10,000 Br. R. T. Thus in 1920 the average size was within the 5,000 to 8,000 Br. R. T.

[1] The article mentions that in the further development of the correction, according to the plan of 1903, upon the demand of Prussia, a weir had to be built in Hemelingen upstream from Bremen so that the tidal district is limited at the upper end.



A=Controlling water slope B=Depth of submergence C=Wake D=Depth below ship E=Excess for unfavorable tides F=Position for bottom

Bremer Null=Datum at Bremen Entwurf=Design

Fig. 125. Longitudinal profile of the lower Weser. Design of 1925

(about 3,200 to 5,200 tons net) group, the draft of which ranged between 7.50 to 9.10 m. (between 24.6 to 29.8 ft.).

In 1924 there was evidence of the necessity of developing the Weser to the upper limit for freight ships of 8,000 Br. R. T. (approximately 5,200 tons net) capacity. This meant developing the river for an average draft of 8.40 m. (27.6 ft.).[1] The essential features of the design are indicated by the profile in Fig. 125. The former widths are no longer sufficient for safe navigation by large vessels, and broadening is planned. The width formerly increased from 70 m. (230 ft.) at Bremen to 150 m. (492 ft.) at Brake. In the future, the width is to begin with 100 m. (328 ft.) at Harbor II, Bremen, and widen to 200 m. (656 ft.) at Brake, so as to reduce the danger of large ships becoming jammed between shores or striking one another.

In Bremen harbor during low upstream stages and prevailing east winds, large ships have frequently had to wait several days for favorable water stages. The bed must therefore be figured for tides which lie .4 m. (1.3 ft.) lower than the average. To this must be added increased submergence due to the suction of the propeller, amounting to as much as .7 m. (2.29 ft.) at Bremen and .4 m. (1.31 ft.) at Bremerhaven, and the amount that the water stage will sink as a result of deepening the river. A further .3 m. (1 ft.) must be allowed for clearance below the bottom. Ships must be capable of leaving or reaching the harbor within a period of two hours, so that several ships may follow each other at different intervals. According to experience, therefore, the outgoing ships must begin their journey from Bremen two to four hours before the high-water reaches the city so that they can reach the high-water peak at Vegesack and Brake. Thus they must be able to start outward at low-water. The interesting necessity therefore arose of dredging the bed deeper at Bremen than at Vegesack. A comparison of the bed-level lines of 1903 (1914) with the 7 m. and 8 m. (23 ft. and 26 ft.) depth designs is of interest in this connection. In the design of 1923 the bed falls from Bremen to Vegesack; in both other designs, it rises from Bremen toward Vegesack. Upon entering, the ships must start from Bremerhaven one to three hours before HW; water-stage-travel lines may be determined accordingly. The water-stage lines for ships having a definite velocity outward and a correspondingly greater velocity inward are indicated in the profile. The results of computations for the lines are superimposed on the profile. It is important for ships of 8 m. (26 ft.) draft to have 9.4 to 9.1 m. (30.8 to 29.8 ft.) water depth, depending upon the width of the channel. Beginning with the first correction, 1889, up to the time of

[1] Extract from a report on the design for improving and deepening the lower Weser for 8 m. (26 ft.) draft sea ships, by Plate, director of the river regulation, June, 1925.

constructing the 7 m. (23 ft.) channel, there has been a total of 59 million cu. m. (77 million cu. yd.) of ground dredged, and a further 10 million cu. m. (13 million cu. yd.) up to 1925. By 1932 it is estimated that 21 million cu. m. (42 million cu. yd.) will be dredged. In November, 1926, ships having a maximum draft of 7.7 m. (25.3 ft.) could enter and ships with 7.8 m. (25.6 ft.) draft could leave.

Ships of 5 m. (16 ft.) draft used to leave Bremen traveling seaward at a velocity of 8 km. (5 mi.) per hour and increased their speed to 12 km. (7 mi.) per hour at Bremerhaven. Since deepening the channel to between 7 and 8 m. (23 to 26 ft.), the ships are allowed to travel at only 6 to 11 km. (4 to 11 mi.) per hour, because of their increased size. The new design indicates that even unfavorable conditions need not deter formulation of a large development.

Widening of the channel sections, a step which was indispensable to navigation, favors the entrance of the tidal wave. After the proposed development has been realized, more tidewater will enter and flow from the mouth than previously. The scouring force will become somewhat larger and the influence of the flow from the upper course will become smaller. It is believed that the ground water will not sink more than 10 cm. (3.9 in.) at Vegesack and 5 cm. (2.0 in.) at Farge. The height of storm tides in the lower Weser is reduced as the size of the receiving basin is increased. No upstream movement of the salt limit is expected.

g. The Outer Mouth District

Definite rules can be set up concerning the correction of the estuary of a river. Experience shows that disregard for these rules results in severe damage to the lower river. The harmful effects are not as severe in the lower course of the river itself as in the outer mouth. There is a definite interrelation between the river form at the outer mouth and the quantity of water which moves into the lower course of the river. The smaller the quantity of water discharging from the lower course, the nearer the sand will deposit to the mainland in the outer district. Furthermore, the less the discharge, the less will be the resulting scouring action, consequently the more unfavorable the outer mouth will be to navigation. The condition at Le Havre shows that this interrelation can not be left out of consideration without serious consequences. There is no doubt that in the course of time laws will be found which are applicable to the outer mouth. The works of recent years by L. Franzius, Kruger, Suling, Plate, Engels, de Thierry, and others in connection with the Jade and the Weser, allow much to be hoped for in this regard. Because of the careful studies made of the Weser, the outer Weser is here discussed, consideration also being given its relation to the outer Jade.

The outer mouth district of the Weser and Jade are directly connected; a change in one is always accompanied by a corresponding change in the other.

The development of river mouths is especially difficult when several outer mouths adjoin one another as is the case in the German Bay. A particularly difficult situation must be coped with at the junction of the Jade and Weser mouths. The occurrences are especially difficult to recognize here, because the period of change extends over several generations; thus, one complete cycle usually does not occur within the experience of one person. Furthermore, the number of sufficiently accurate sea maps of past periods is inadequate. Systematic depth measurements in the North Sea have been made for only one hundred fifty[1] years; occasional maps were made since 1680.

At the present stage of our knowledge, it is impossible to set up general rules or laws regarding the changes in outer mouths. Hence, the outer Weser[2] is taken as an example to show the order of occurrences in a particular case. For other cases, the necessary information may also be obtained by long periods of observation.

The wandering of the sands is of special importance for the mouth district of the Jade, Weser, and Elbe. The sand comes from the direction of France and Belgium in a zigzag path along the coast. It penetrates into every river mouth with the tidal wave and wanders out again with the ebb current. The effect on the islands has been observed for many years. The island Wangeroog borders the Jade district at the west. Sand masses are periodically loosened from here in sheets, and then wander from Minser Oldeoog through the Jade and Weser toward the Elbe. Natural rises below water doubtless play a rôle similar to the islands above water around which the sands settle, and from which they become loosened when the masses have become large enough. Currents and wandering sands are cause and effect which reverse from time to time. The currents (in conjunction with storms) are the cause of sand migrations, but (just as a man who carries a heavy sack can finally stumble over the sack) eventually the sand heaps become too large and close the way to their own current, forcing it to change direction. Thus instances may arise in which currents entirely disappear, reverse, or undergo other changes.

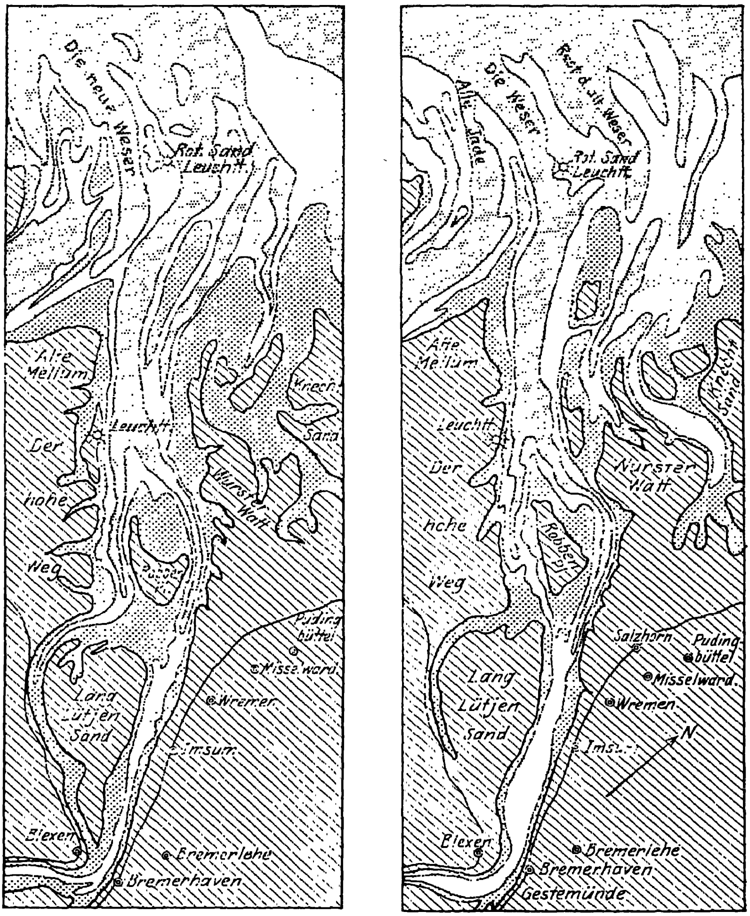
A comparison of the four diagrams of the outer Weser (Figs. 239 to 242) for the years 1859, 1889, 1899–1900, and 1921, respectively, show the changes which are occurring. Still older maps, including the years 1680, 1791, and 1812, are available. However, the time interval between

[1] Carl Woebcken, *Deiche u. Sturmfluten*, Friesen, publisher, Bremen, 1924, p. 36.

[2] Plate, *Bremen und die Aussenweser*, Werft, Reederei, Hafen, 1923, Vol. 1 to 3.

them is very irregular. German marine maps have been made periodically since 1859.

As already mentioned, the sands which become loosened from Minser Oldeoog wander northeast over the Jade into the river channel of the outer Weser. Ebb and tide cross each other at about 10°. The sand

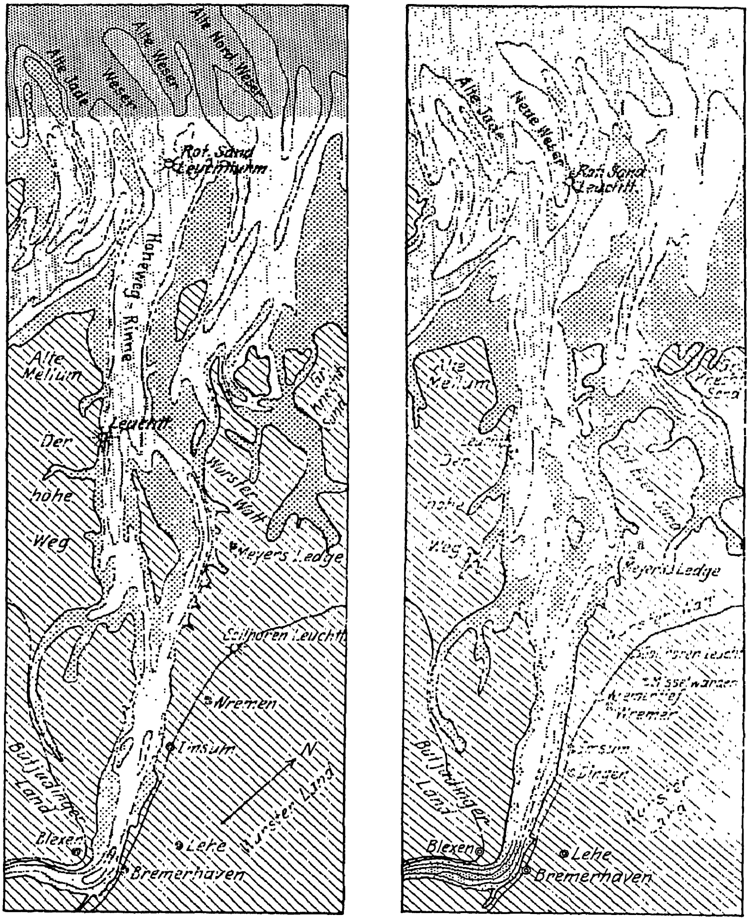


1859 1889

Figs. 126 and 127. Outer Weser 1859 and 1889

first joins the north end of the old Mellum Flats, which form the west limit of the Weser. Here it remains until it widens the Mellum Flats to the extent that river channels begin to be gnawed into the flat by the current. Then large quantities of sand become loosened again and wander further, thus forming a new western mouth to the Weser. The

four maps show clearly the changes which have taken place. In 1859 the red sand which was built up at the famous Red-Sand light tower, loosened itself from the Mellum Flats, so that by 1889 the former Weser east of "Red Sand" moved to the west side. Consequently by 1889 the "Red Ground," which was still west of Mellum in 1859, had joined



1899/1900 1921

Figs. 128 and 129. Outer Weser 1899–1900 and 1921

Mellum. (The sands retain their name until they reach the range of a former sand mound; then they take the name of the sand replaced.) From 1900 to 1920, another break of the connection between Mellum and Red Ground took place. A navigable channel developed as an entrance to the Weser east of Red Ground (which had wandered toward

Red Sand), and extended to the old Jade (one of the Jade mouths). The occurrence is recognizable as far back as 1791, and may be traced back as far as the year 1642, since in the latter year a sailing guide announced the displacement of the channel from the east Weser to the west Weser. Inasmuch as ships of that time required only a small draft, the obstruction must have been very pronounced. Plate estimates that it requires between sixty and seventy years for a complete cycle. The forces are of such a nature that at present we cannot expect to hinder the occurrence enough to keep the mouth arms in position. The most that can be done is to obtain an accurate understanding of the occurrences.

Relatively bad or good conditions are attained in the lower river, according to whether one or several well-developed mouth arms exist in the outer mouth. The conditions of 1859 and 1921 allowed a strong penetration of the principal tidal waves into the Weser. In 1889 and 1900, on the other hand, the wave was very much obstructed by the sand, and secondary waves entered, which weakened the main wave. The development of the channel further inland usually determines the navigability of the outer mouth. It is of importance to know, however, that this channel is greatly affected by the development of the actual outer channel.

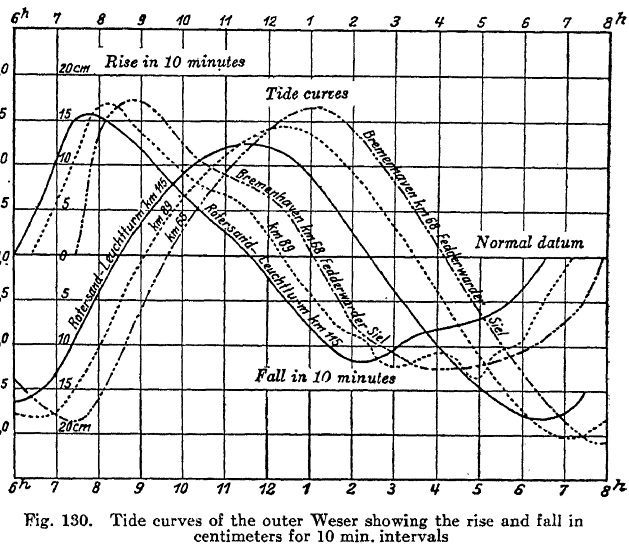


Fig. 130. Tide curves of the outer Weser showing the rise and fall in centimeters for 10 min. intervals

Further landward between Bremerhaven and the Red-Sand light tower, three channels in the Weser are distinguishable; two are principal channels, namely, the Wurster Arm which lies entirely in the North Sea, and the Fedderwarder Priel which lies entirely in the southwest side. Between the two lies Wremer Loch, nowadays also called Fedderwarder Arm. The Fedderwarder Priel was the principal navigation arm for the Weser as early as 1680. At that time a fork developed, which resulted in this passageway being discarded in 1825 and replaced by the Wurster Arm.

The old principal navigation route then became silted up and completely closed. The Langlutjen-Sand became joined with the mainland on the west, so that the Fedderwarder Priel is now readily recognizable only on the northern portion. The Wurster Arm maintained itself very well for many years, but by the end of the past century improvement had to be made with the aid of brush-work structures. At that time the course of development could not yet be clearly recognized, and it was attempted to maintain the Wurster Arm. A particularly unfavorable location in the Wurster Arm was the Dwarsgat (Cross-door), which showed pronounced bar formation. The development of the Robben Flat was particularly important in this connection. In 1859 it was broad and very blunt on the north side. It continually became narrower, but its northerly point wandered northward so that the large quantities of tidal water flowing from Wremer Loch obtained a better discharge. The scouring effect in Wremer Loch was thereby increased. The latter deepened and by 1921 was larger than the Wremer Arm. Fig. 129, which is based upon extensive measurements, very clearly shows the development of the Weser.

The following summary of the cause of the occurrences is presented by Plate:

The causes of favorable development of the Fedderwarder Arm are as follows:

1. The opening of new arms at "Red Sand."
2. Rapid fall of the water in the Fedderwarder Priel after the drying of the tidal lands, and thereby increased fall in Wremer Loch during the last third of the ebb.
3. Shorter length of the Fedderwarder Arm compared to the Wurster Arm.
4. Northward wandering of the Robben north-steert due to sharp attack by ebb current and strong transverse current toward the north.
5. Improvement of tidal flow as a result of this deposit.
6. Reduction of the Robben south-steert, and thereby increase in discharge from the Wurster Arm.
7. Increasing difficulty in the use of the Wurster Arm.

The latter arose because of the following circumstances:

1. Forward penetration of the Tegeler[1] tidal wave to Dwarsgat.
2. Bar formation in the Dwarsgat at Meyerslegde and Solthorn, and the formation of intermediate deposits due to unfavorable current conditions and the criss-cross flow of ebb and tide currents.

The task of maintaining the Wurster Arm was unsuccessful because of the unfavorable circumstances. The gigantic brush-work structures

[1] The Tegeler Arm is a northern secondary arm toward the sea.

in the Weser frequently became completely destroyed. The effort to maintain the Wurster Arm gave valuable experience. It is certain that well-arranged structures might assist in such a development, but not permanently.

In 1923 upon the basis of experience gained, it was resolved to give up maintaining the Wurster Arm and to further the development of the Fedderwarder navigation channel. The channels to the Fedderwarder Arm had been scoured to such an extent that only transition bars in the Wremer Loch had to be broken through to a length of 3 km. (2 mi.) and 200 m. (656 ft.) wide. The Langlutjen Arm was weakened by a groin. A separating dam on the south-steert of the Robben Flat was also built. In the future, it is planned to build only river structures to protect the navigation water from becoming unruly. These should also promote deepening of the channel. It is planned to create a channel within five years which will have a width of 200 m. (656 ft.) and a depth of 10 m. (33 ft.) below LW [13 m. (43 ft.) below HW]. While formerly all of the work was done by the free city of Bremen, the German Imperial Waterway Department undertook the task beginning April 1, 1921.

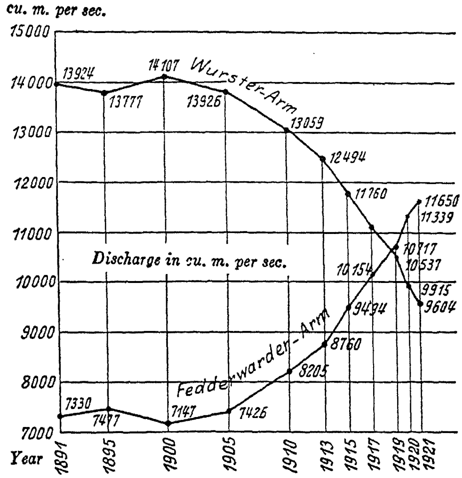


Fig. 131. Change in water discharge of the Weser during half ebb condition in the Wurster and Fedderwarder Arms

PART THREE

EFFECT OF THE SEA ON THE COASTS; SEASHORE DEVELOPMENT; LEVEE CONSTRUCTION

A. NATURAL CHANGES IN THE SHORE

a. Cause of Changes

Harbor development requires a knowledge of the nature of the shores because it frequently becomes necessary to change the shore in the construction of a harbor. The natural form and stability of the shore, and the possibility of artificial transformation, greatly influence the design and construction of a harbor.

All seashores are constantly undergoing transformation, either geologic or local in character. Geological changes consist particularly of raising and lowering shore lands and are of great significance. The long-continued sinking of the German North Sea coast, which probably is accompanied by an increase of the tidal range on the German coast, is the cause of the highly destructive influence exerted by great storm tides. Although the levees were continually sinking, this was without the knowledge of the marsh inhabitants.[1] Local causes are less impressive; either land is washed away so that the shore breaks down, or sediment is deposited in front of the shore causing land formation. Both conditions depend upon the influence of waves and currents. The effects of waves and currents are influenced by the form of the shore and its position with reference to the currents. As a rule, stretches of land which project against the current become eroded. Bays, on the other hand, are subject to sedimentation. Under similar flow relations, the magnitude and nature of the changes depend upon the nature of the ground, conformation of the shore, and also upon the climate.

All influencing factors must receive adequate attention. The height of levees must correspond to the possible storm tide heights. Levees must be raised in accordance with requirements, and local changes must be carefully studied. The latter problem is twofold. At some points, sedimentation must be aided, at other points hindered, depending upon the controlling interests of land development and sea navigation.

[1] Carl Woebcken, *Deiche u. Sturmfluten*, Friesen, publisher, Bremen, 1924, pp. 46–48.

The principal attack upon the shore is due more to waves than to currents, the latter usually attacking the shore only at great storm velocities. Currents act more or less indirectly in that they transport the loosened material to locations where it may again be deposited. On the other hand, waves displace material principally in the immediate neighborhood, while currents continually carry it away. For example, it may be assumed that a sand-transporting current continually flows from Belgium to Jutland.[1]

If a wind is directed toward the coast, waves are driven landward. Due to the progress of the wave, water is also moved toward the land. This generates a backing up of water at the shore, the water intermittently flowing seaward as an undercurrent. The undercurrent transports the bed material, which has been loosened by the surf, away from the shore toward the sea to a depth at which the return flow ceases. This breaking down of the actual shore line is counterbalanced by deposition which occurs in a forezone lying further seaward. The wave exerts considerable force on the ground before it breaks. This force is directed toward the shore and pushes various grades of material slowly before it, the size of particles and quantity of material depending upon the strength of the wave. At the limiting line, where the return current and the current of the bottom of the approaching wave equalize each other, bed materials from both sides come to rest and may form a ledge; that is, a mound under water some distance out from the shore. The ledge formation here described develops only for strong wave motion. For mild waves the erosive action is not great and material is moved upward only in the forezone. If a strong coastal or wave current prevails in connection with a mild attack of waves, the rate of erosion or deposition of sediment may be increased. A coastal current of this nature prevails in the south Baltic Sea and flows eastward. As a result of the strengthening action of the wind, in several localities it may reach a velocity of as much as 3 m. (10 ft.) per sec. In case of opposed wave motion the current may be entirely stopped at times. On the southern coast of the North Sea the drift currents behave similarly, because they are dependent upon the prevailing west winds.

Tidal currents often affect the shore in a manner similar to that of the regularly changing coastal streams. In some localities they flow in along one line and out along another. This causes a continual breaking down of some shores and building up of others. If the tides flow back and forth along the same path, the effect is of no consequence. For sea currents, just as in the case of continually flowing streams, the amount

[1] Krüger, *Z. Bauw.*, Vol. 7, 1911.

of material and diameter of particles transported depends upon the strength of the current. (See experiments by Engels.[1])

The bed-building action of a river through deposition of sand and gravel in the sea is treated in a particular section because of its vital importance.

b. Steep Shores

Steep shores consist not only of rock cliffs but frequently of earth and calcareous masses bound together by clay. If great water depth exists immediately in front of the steep shore, it may be concluded that the shore consists of a hard rock formation. A strand, although quite narrow, is an indication of soft material in the steep shore. The more nearly vertical the shore and the harder the material, the greater is the probability of maintaining the steep coast. At such shores, waves oscillate rather than cause impact.

Atmospheric influences are of great aid to the action of the sea. The preliminary work of destruction is performed by atmospheric influences. Without these the sea would require several times as long a period to produce the same destructive action. Weathering conditions are extraordinarily strong in northern Germany. The sun warms the cliffs to high temperatures on hot days, but irregularly, depending upon the form of the surface area. Consequently, numerous temperature cracks are formed. The water penetrates these cracks and wedges rocks loose in the winter through freezing. The sea completes the destruction by heavy wave impact during high tides. The loosened ruins are then pushed back and forth by the waves until sufficiently ground to be washed away by the current.

If the shore is not very hard, the breaking down proceeds so rapidly that a shallow strand is formed in front of the shore. The strand acts as protection to further disintegration at times of low tide. At high tides the strand directs the waves mildly upward and thereby develops their impact force to the greatest extent, so that the strand becomes harmful during periods of high tides. The waves break with such force that even heavy stone ruins may be thrown against the wall of the shore. Under these circumstances, the cliffs are first gnawed and washed away at the foot, until large overhanging portions are formed which finally break off under the action of frost and their own weight. Where there is no frost action, as in the tropics, its effect is partially replaced by the greater heat.

If the soft shores contain horizontal, water-carrying sand layers, the breaking down proceeds particularly rapidly. The sand is scoured out,

[1]Engels, *Zentralbl. Bauverw.*, 1908.

the clay or chalk layer sinks, breaks, and is torn down by strong wave attacks. The first protective measure for such shores consists in carefully conducting off all spring water. Layers of coarse gravel and erratic block are just as harmful as sand.

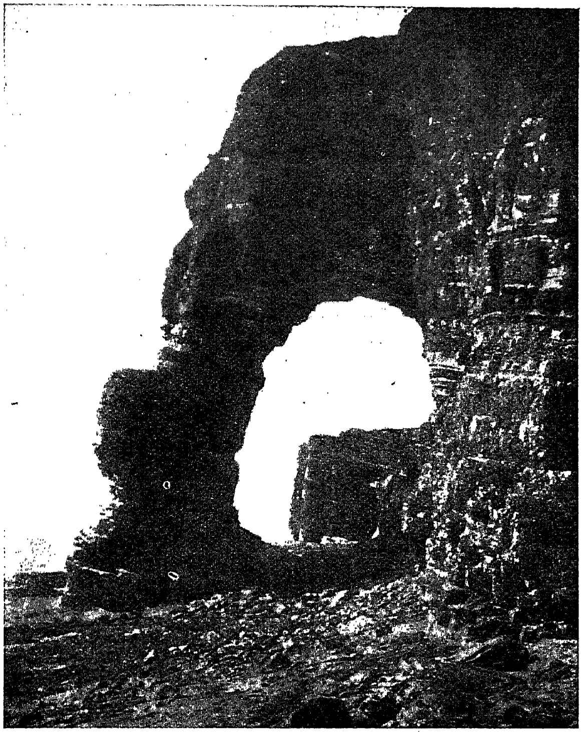


Fig. 132. *Schnepfgatt*, a natural portal on the shore of Heligoland

An instructive example of the manner of destruction of steep shores is given by the southwest coast of Heligoland. Here large caves have been washed out in places where softer stone portions existed in vertical crevices. The caves gradually develop into portals which are open from end to end. The top of the portal continually becomes smaller by the breaking down of parts frozen off or knocked off by water, until eventually it breaks down under its own weight. Individual columns remain subject to the destructive action until they completely disappear. Fig. 132 shows Schnepfgat, a cave of the Heligoland shore shortly before breaking down, 1895. Capri indicates similar occurrences; a natural arch also stands there as the remains of an old cliff.

c. Flat Shores; Reefs; and Dunes

Changes in flat shores take place much more rapidly than changes in steep shores. Ledge formations result from a breaking down of the shore toward the sea, while strands and dune formations result from material being brought to the shore from the sea. By strand is meant the (usually sandy) shore strip which is frequently overflowed and on which no substantial plant growth occurs. The actual building material for the

strand is sand, but also rubble from ground-up flint, limestone, and other materials. Sand is scattered along the coast in a broad belt, often beginning at an elevation of 12 m. (39 ft.) below the surface of the sea. Underwater reefs, which consist principally of sand driven up by the sea, are formed seaward from the strand. A reef slopes gently toward the sea and slopes at a definitely steeper grade from its crest toward the shore. The sand is driven over the gentle slope and deposits in the back at its natural angle of repose (Fig. 133). Reefs are of greatest importance in connection with harbor entrances. Frequently after heavy storms, reefs arise which temporarily close the entrance to navigation for deep draft ships although they soon disappear by the action of other winds. On the North Sea coast, for example, reefs develop to heights up to 2 m. (7 ft.) over the bed. Hence, it is important to place harbor entrances as nearly as possible in such places where dangerous reefs will not form.

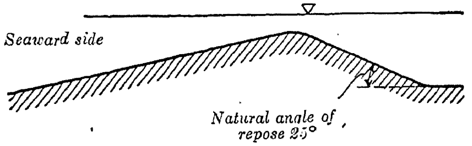


Fig. 133. Sand reef. Mild slope toward the sea and natural angle of repose toward the land

Frequently several reefs occur one after another, depending upon the strength of the sea forming them. The last reef may be considered as the strand itself. The strand is flat, practically horizontal (1: 50 to 1: 100), where both the shore and sea bottom consist of sand. It is steeper (1: 5 to 1: 20) if the sand belt is quite narrow. In flat strands which receive sufficient additional sand through the waves, the sand may lie dry at low-water stages and be driven landward by the wind. The wind thus forms sand reefs just as the water does; however, those due to the wind consist of dry sand and are called dunes. Dunes are of importance in harbor construction in connection with shore protection.

The sand eventually encounters sufficient resistance to stop its movement at some particular location, so that the following sand quantities must be driven over it. On the seaward side a gentle slope arises which is flat enough to allow the wind to drive sand up over the mound. On the back or landward side, there is a steep slope at which the sand rests at its angle of repose. Sand dunes, like reefs, may occur in a series of several. When they are stabilized by growths of oats and grass, they protect the shore against too extensive breaking down by storm tides, but continually require renewal by sand blown up during quiet weather. Where strong tide or drift currents keep the strand narrow, so that no sand is available for the renewal of the dunes, the dunes can not be made permanent nor can they be depended upon for permanent protection of the shore.

d. Dune Protection and Culture

The cost of maintaining dunes is low in comparison with that of artificial structures. Well-maintained and sufficiently large dunes are seldom entirely devastated by storm tides. They are reformed very rapidly by nature when given artificial assistance.

The manner of protecting the foot of the dune at narrow strands will be explained in the discussion of protection works. Usually this work is required only for narrow strands.

Too little attention to stabilization results in wandering dunes. The maintenance of dune plants requires the continual addition of sand, blown up from fresh strands, which is still saturated with nourishing salts. Where this addition is not supplied, the plants gradually die off and the dune becomes bare. The wind which formerly was the constructor and maintainer of the dunes, then becomes its worst enemy. It drives the dune sand, which is scoured out by the rain and has lost its binding force, up the seaside slope so that it is either blown further

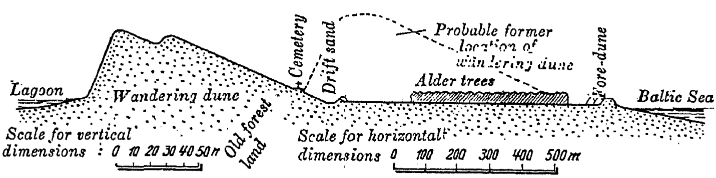


Fig. 134. Sand dunes

inland or slides down the steep slope of the steeper landside. The dunes are rolled further inland; that is, they wander (Fig. 134).

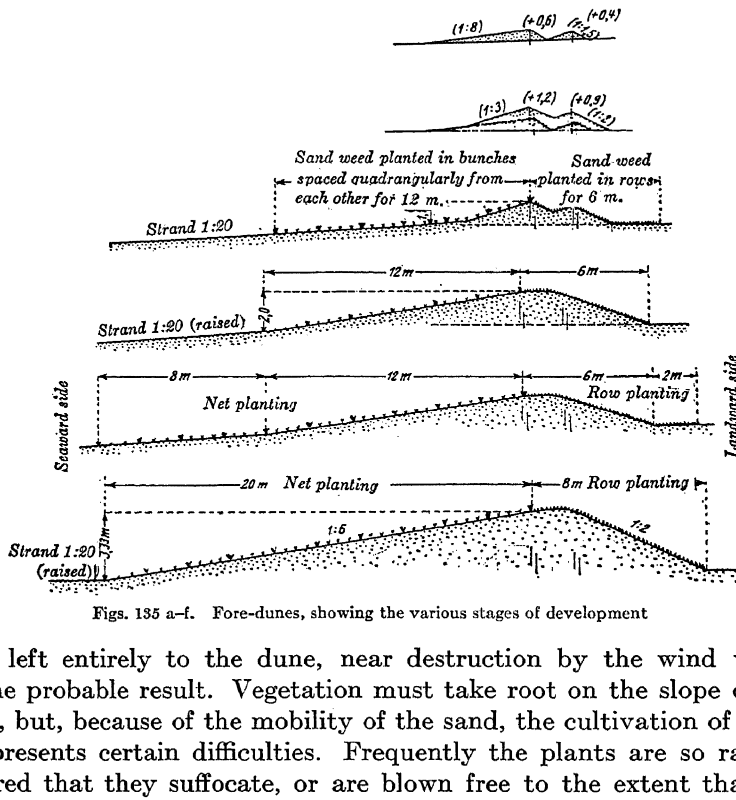
In this process all vegetation in the path of the dune is destroyed. In East Prussia, even forests have been penetrated and suffocated. The wandering may amount to more than 5 m. (16 ft.) per year. Breaking off of the unprotected strand usually takes place as far inland as the dunes wander from the sea.

The principal problem involved, therefore, is the growing of vegetation on the dunes and maintaining this growth or providing new vegetation. The preliminary problem is the creation or maintenance of a source of nourishment for the dune plants on the strand.

A particularly effective means for nourishing dune growth consists in creating fore-dunes in front of the older ones. This is done in the following manner: Flexible poles without cross brush work are placed along the strand in several rows of about 5 m. (16 ft.) intervals. These brush fences are set in slender lines without corners and are cut to a uniform height with garden clippers. They extend .8 to 1 m. (2.6 to

3.3 ft.) above the strand and must allow passage of the wind, causing its velocity to be somewhat diminished. Dense fences are incorrect because they cause the opposite effect; namely, increased and upward deviation of the wind velocity.

In the protection of these brush fences, the sand falls till the fence is completely covered with sand. The fences must then be renewed on the developed dunes until a definite height is attained. If further results



Figs. 135 a–f. Fore-dunes, showing the various stages of development

were left entirely to the dune, near destruction by the wind would be the probable result. Vegetation must take root on the slope of the dune, but, because of the mobility of the sand, the cultivation of plant life presents certain difficulties. Frequently the plants are so rapidly covered that they suffocate, or are blown free to the extent that the roots come to the surface. A preliminary cover consisting of dead vegetation has been found necessary. Next, a cover of pine tufts is laid, with a brushwood protective cover, or new flexible bush fences are placed, under the protection of which young plants can grow.

Only a few kinds of plants can survive on dunes. Some of the best proven types for artificial planting are strand oats, strand wheat, and strand barley. Planting with seedlings should preferably be done in the spring or late autumn. They are placed at protected locations in tufts planted at equal intervals in staggered rows on the windward side. On

well stabilized dunes the usual fir, birch, and alder trees are planted as culture plants in the dune valleys.

The growth of willows should be hindered on the fore-dunes because they usually cause irregular sand funnels by concentrating the wind attack. Every rigid obstacle on the windward slope of a dune has an effect similar to that incurred by a "dune" in water; it generates a wind scour. The destruction of entire sand dunes frequently starts with such wind funnels. The successful generation of fore-dunes is one of the greatest successes of modern dune nursing.

Figs. 135 a to f show sections of the artificial formation of fore-dunes. Old dunes which have become bare may be vegetated in the same manner as fore-dunes.

The presence of dunes does not do away with the necessity of building levees behind them. However, the levees may be made considerably weaker than if no dunes lay before them.

e. Bays and Islands

1. Bays

If an ocean bay extends inland at a sharp angle to the ocean current, only the edge of the current deviates somewhat into the bay. Because of the resulting diminution of velocity, the current allows its sedimentary load to fall in narrow streaks as tongues of land. Fig. 136 shows such formations in the tongue of Hela and the Courish Peninsula. Such tongues have often arisen as a result of the sinking of the coast where a slender body of ground lies higher than the neighboring land and therefore towers above the water as a tongue of land. These peninsulas may become so long that they practically close the bay. Such bays are called haffs, lagoons, etc. At least one opening to the sea remains, because during changing stages of the sea, water flows in or out of the bay in the process of equalizing the water elevation. Likewise, an opening must serve for

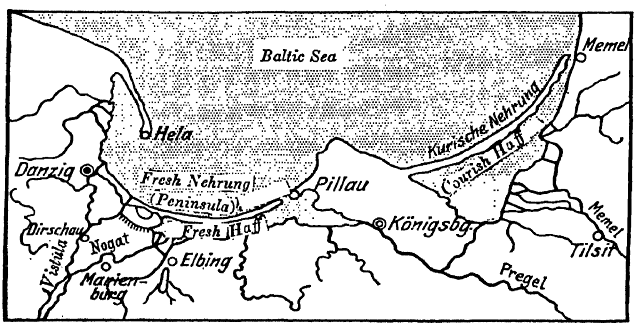


Fig. 136. Courish Peninsula (Kurische Nehrung)

the further movement of the river water coming from streams which discharge into the bay. The currents in both instances hinder the formation of large sediment deposits in the opening. If the coastal current alternates daily, for example, as do the tidal currents, then several openings will remain. This is true of the East Frisian Islands.

Bays of this nature, especially those with one opening, require special consideration because their shores and they themselves behave in an individual manner.

Closed-off bays, to a much greater extent than open bays, are strongly inclined to build up land. At times of storm the sea water enters the opening of the bay as a strong current. Because of the greatly increased cross-sectional area inside the entrance, the water soon comes to rest and allows the suspended sediment to deposit. If the sediment in front of the peninsula is stirred up it will also be carried through the bay opening by the current. The formation of land is especially favored on the mainland side because of the inflow of rivers. There is no coastal current in the bay and hence the entering sediment is not carried further. In addition to the general building up of land, extensive delta formations are not uncommon. The delta of the Nogat in the Fresh Haff is said to have formerly moved forward at the rate of 40 m. (131 ft.) annually.

If a sea harbor lies in a lagoon, maintenance of depth in the fairway is the most vital problem to be solved. Even if the fairway is permanent to a certain extent in both depth and width, it is frequently necessary to prolong the sea side of the opening by guide dams. They centralize the scouring force of the stream in the gatt and conduct the navigable channel to deep water.

2. Islands

Most islands lying near the coast of the mainland have been formed by the breaking down of land. Many coasts sink slowly and the process cannot be checked. If points of the coast lie higher than the land behind them, in the course of time the land areas which become submerged are gradually scoured out until the higher shore is entirely separated from the mainland. The former shore remains in the form of islands. The submerged land between the islands and new shore in many localities emerges regularly as tidal land at low water. The procedure in which the sinking of the coasts took place has not been explained up to the present time, but it is probable that the process did not take place regularly either with regard to time or height. An assumption which is in all probability correct indicates that the North Sea coast formerly reached far out into the sea, about as far as the Dogger

bank (Fig. 137). At that time England was still connected to the continent. The Rhine flowed through the "Silber-rinne" (silver channel) into the much smaller North Sea and the Elbe had its own mouth further northward. Sinking has taken place to such an extent that the former land no longer emerges at the time of low water. Heligoland is probably the last island of this former Continental area and remains probably only because of its rocky character, though it was formerly a large, rich, thickly populated marsh island.

Sandy islands and marsh islands are differentiated from each other according to the make-up of the ground. Dunes are found on both types. The islands show a very different reaction to storm tides.[1]

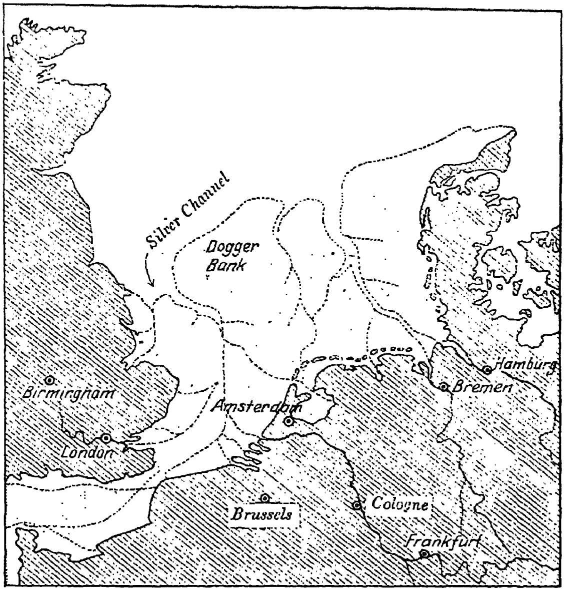


Fig. 137. The probable former limits of the North Sea during the early post-glacial period

No continual decrease in size of most sandy islands has been noticed within historic times but (as for example, in the East Frisian Islands) a wandering of the islands is noticeable. The East Frisian Islands are strongly attacked on the west side, and usually the sand which is torn loose again becomes deposited near the island at the east end, since the ebb current is not strong enough to carry this heavy material very far. Local changes occurring in these islands are just as dangerous to the generation implicated as complete destruction. It is probable that formerly sandy islands also existed along the coast. Today the form of such islands, provided shore protection is practiced correctly, can be made permanent at least for a period of several generations. The island Wangeroog is an exception to the possibility of

[1] Fülscher, Uber Schutzbauten zur Erhaltung der ost- und nordfriesischen Inseln. *Z. Bauw.* 1905; Krey, an article having the same title as the foregoing one, *Zentralbl. Bauverw.* 1906.—Quite valuable; Plener, *Zeitschr. f. Arch. u. Ing.* 1856, p. 44, Hanover, which stands in strong opposition to Fülscher.— Krueger, *Z. Bauw.* 1911, Vols. 7–9, with good maps.

providing semi-permanence. During the storm tide of December 1854, a town of fifty houses at the west side of the island was destroyed by waves. The chain of dunes and fruitful clay ground were washed away so that only a bare strand remained. The island became 2 kms. shorter. There are indications that other islands have been subject to similar occurrences. In 1924 at the extreme west end of Baltrum, a graveyard was laid bare which formerly doubtless was in the middle or east end of the island. Three hundred years ago Juist consisted of two parts, there then being an island between Juist and Norderney.

The protective action of islands for the continent is questionable. They usually lie at such a long distance from the mainland that the waves are really not significantly reduced. Frequently the foreland in front of the continent dike has been badly torn away just where (according to earlier views) the island protected it most. The principal value of the island rests in its bathing beaches. Without these (based upon the fruitfulness of the ground) it would be valueless, as not even trees will grow on most of these islands. Norderney with its small forest is an exception.

Conditions at the North Frisian marsh islands are similar to those at Wangeroog and Baltrum. The solid clay ground of these islands can undergo the wave attack longer than sandy ground, but after breaking down, the clay is transformed into mud and washed long distances by the current. Breaking off of marsh islands has proven to be very pronounced. Based upon present day values it has been estimated that probably billions of dollars worth of land has been devastated. The entire island group, Nordstrand, Pellworm, and Föhr, once formed a large fruitful island with dozens of communities. It was destroyed by the storm tide of October 11, 1634, with a loss of thousands of people and animals. Protection of sandy islands is unquestionably necessary. The maintenance of marsh islands is a national duty to the same extent as protection of marshes along the coast. Maintenance of the Heligoland rock island with its dune island was necessary from a military point of view in order to insure Germany's status as a sea power. At present the bathing beach on the dunes is of most importance.

The maintenance of the German islands along the north coast is for the common interest of navigation. The North Sea coast of Germany is one of the most navigated and also one of the most dangerous coasts in the world. Numerous light towers erected on the islands diminish dangers significantly and indirectly pay for themselves by diminishing the number of shipwrecks.

B. ARTIFICIAL ALTERATIONS OF SHORES

a. General Considerations with Reference to Protection of Shores against the Attack of Water

The need of shore protection structures is dependent upon the value of the endangered land and the seriousness of the danger to the coast threatened. Where islands show themselves as protective to the coast by diminishing the wave attack, the structures protecting the islands must be considered as protective measures for the coast.

Permanent protection is required particularly for highly productive marsh lands, regardless of whether they are on islands or the continent.

Just as in river control, distinction is made in adjacent protective works, parallel works, and groins. Adjacent protective works may simply guard against further breaking down, while projecting works may cause further deposition and building up of land. Adjacent works form a resistant cover to the shore; projecting structures extend beyond the shore, in the form of parallel works or groins, far into the forestrand and force the deposition of mud and sand during middle and higher water stages.

The more rapidly and extensively the protective structures are developed, the more effective they become. During a single storm tide, the property destroyed may have a higher value than the cost of all of the protective structures which would have been necessary to avoid the destruction. It is believed that in the Marseilles tidal flood of 1362, more than fifty parishes were destroyed. Thus, if it is possible to provide security to a large district for several years with light and cheap construction, this will be preferable to protecting only a portion of a district permanently with heavy structures and allowing the remainder to be unprotected against storm floods until further means are placed at disposal. Permanent protection may be developed later. If the lighter works do not provide certain security even for a few years, then it is better to construct as much permanent protection as possible with the available means.

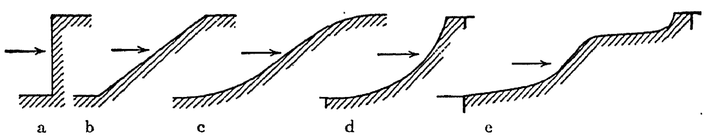
If the attack of recessed shore stretches can be diminished by protecting projecting land corners, even if only through large expense, it is better not to wait until after the breaking down of the corner, because then the shore itself will have to be protected by structures which are usually even more costly. The same is true with regard to protection of strands having dunes lying in back of them. Where coastal currents endanger the strand, it is often permissible, even at large expense, to construct groins extending far out into the sea so that dangerous stream channels will not be gnawed into the coast and destroy the strand and the neighboring dunes.

Where the building up of land progresses rapidly and strong attack of waves does not occur, light wood or brush-work structures with rock packing may be erected. These structures will soon become imbedded in the sand to the extent that attack will no longer occur. Where deposition proceeds slowly, heavier rock layers on elastic straw, brush work, or sea-grass underlayers must be used. The stones are usually held fast by thin piles driven into the ground.

Instead of stone blocks, in recent years, the use of concrete blocks or concrete shells stamped in place as one piece is prevalent; the latter, inasmuch as no cracks or crevices are present, suffers a minimum amount of attack from waves. Where the erosion is particularly pronounced, cover works used alone do not have great permanence; they soon become underscoured and cave in. Cover works give permanent protection only after groins have been constructed.

b. Adjacent Protection Works

These works are designed to provide direct protection against the attack of waves and currents to the shore, an elevated portion of the strand, or the foot of dunes. Their actual resistance against waves depends first on their form and second on the heights, respectively, of



Figs. 138 a to e. Forms of cover works

a Steep flat type b Flat type c Type having concave bottom and arched top d Completely concave type e Combination type

the foot and head of the cover works. The influence of the height is very important. The deeper the foot and the higher the head, the greater will be the security, but also the higher the cost.[1]

Cover works are divided as follows:

1. Covering for low shores with adjoining shallow water, the shores lying deep enough below high tides so that the tides are not harmful to them.

2. Covering for high shores lying beside shallow water, the shores being subject directly to the attack of waves.

3. Covering for deep, steeply sloped shores subject to the attack of strong coastal current but with little wave attack.

[1] With regard to the construction of protective works see doctorate dissertation by Hansen, Technische Hochschule, Hanover, 1921.

For shores 1 and 2, in which wave attack predominates, the works are designed in accordance with one of the five types shown in Fig. 138. A choice may be made by reference to the following considerations:

Form a, consisting of a vertical wall, is usable in rigid foreground. It is constructed of wood, rock, or concrete, and requires the least amount of material. The wave surges directly on the wall, so that it is not applicable in soft ground without bed covering because the wall will readily overturn due to underscour.

Form b, a flat incline, is most convenient, but applicable only in case of weak wave attack. The upper portion of the slope is very strongly attacked, waves spring back heavily, beat on the lower corner of the bed, and cause scour at that point.

Form c, a section concave at the bottom and convex at the top, conducts the waves better, is less attacked than form b, but conducts more water up the shore so that the top suffers as much damage as b.

Form d, an entirely hollow section, is no more difficult to construct than c but forces the waves to fall back on themselves. Experience shows this type to be attacked less by waves than forms b and c.

Form c has been improved by making the upper part run horizontally for a little way and then top off with a portion similar to form d (Fig. 138 e). This form is now very much used on the islands of the North Sea. It was first used on Borkum Island, and then on others, including Norderney. It is cheaper than a steep shore protection, and more reliable than profile d which was used on Wangeroog. In all cases it is advisable to protect the shore at the top with a dense cover for a considerable stretch behind the edge. Inclined stone-covers should be used only at clay shores with gravel underfill laid directly on the shore. For this type on sand shores, either thick layers of clay must be placed under the gravel (in cases of weak wave attack), or protection against underscour must be assured by using brush-work underpacking and wattlework forelayers. It is noteworthy that for cover works of this nature a concrete mix of from 1 : 9 or 1 : 10 has been used without addition of coarse gravel or large quarry stones.

Form a was constructed of concrete as a simple shore protection wall in the Admiralty Harbor of Dover. The foreground consists of resistant cretaceous rock.

On the Baltic, several such walls have been constructed. The wall shown in Fig. 139 is a good example of type a in hard foreground. The wall type in Fig. 140 is good for a solid strand which contains soft layers. In both types, the shore behind the wall is raised to the top edge of the wall. Careful provisions have been made for carrying off water from the background through drain pipes, or gravel layers

in both cases. The construction of these drains is a necessary provision for the long life of the wall.

Examples for form b and combination of b and d are given in Figs. 141, 142, and 144. Fig.141 shows the cover works on a sand strand not subject to serious wave attack. Here, as for the following cover works, particular attention had to be given to providing protection against the sucking of sand through the cover layer. For this reason either a dense concrete cover or a fascine sublayer was built into the work.

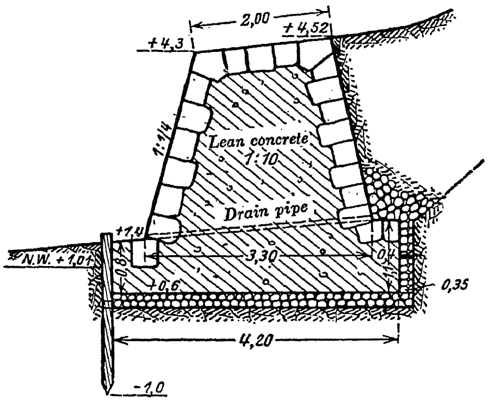


Fig. 139. Shore wall suitable for very stable strands

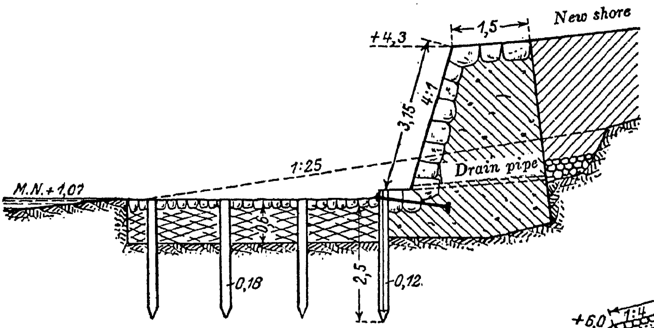


Fig. 140. Shore protection suitable for stable strand having soft intermediate layers

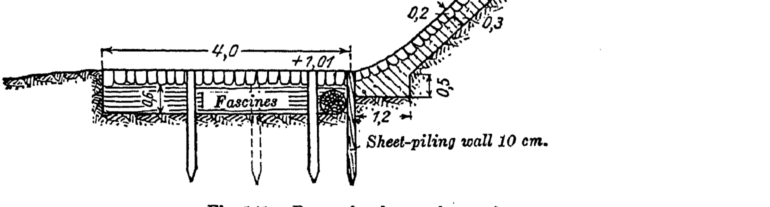


Fig. 141. Protection for sand strand

Without such provision a protective cover would soon cave in, as a result of underscour, even though the stones be set with cement.

Drainage should also be provided for cover structures on sand strands. Such protective works have also often been constructed of concrete without facing. This was done in Belgium.

Fig. 142 shows a well-constructed protective cover on mattress used at Bremerhaven. In Fig. 143 a cover having a water buffer is shown for comparison. The rear water bed was formed by simply constructing the shore protection some distance in front of the foreland. The back cover required for this type makes the construction more expensive. The action of the water buffer, which is intended to catch the overtopping waves, was formerly considered especially advantageous. However, the water buffer cannot be expected to be extraordinarily effective, because in many instances the overtopping waves beat through the thin water cushion as though it were not present. This type of structure is therefore not to be recommended. Hansen states, as a result of experience on the Jutland west coast, that the water buffer is ineffective.

In the Baltic Sea, stone walls made up of large blocks have been used with great success. Of late these have been laid on fascine layers. Construction in rigid layers, that is, direct stressing of the heavy outer stones, is necessary; but squeezing outward with small stones is harmful. It is considered advantageous to have a portion of the wave penetrate the wall, because the strength of impact is lessened thereby[1] (Fig. 145).

A combination of b and d is shown in Fig. 144. This shore cover serves for the protection of dunes on Wangeroog. It is bent over at the top in order to hinder attack at the foot of the dune. The wave breaks on itself because of the back current generated thereby. The foot is further

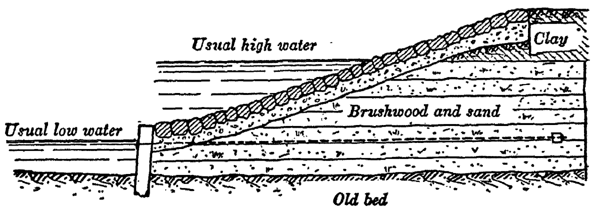


Fig. 142. Section through cover works at Bremerhaven

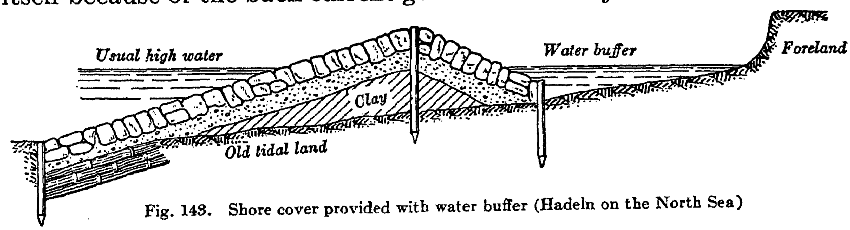


Fig. 143. Shore cover provided with water buffer (Hadeln on the North Sea)

protected by mattress sublayers between a series of piles. This type of shore protection ranks among the best.

The protective cover at Borkum, Fig. 146, is an intermediate of the

[1] For further information, see *Zentralbl. Bauverw.* No. 26, 1908.

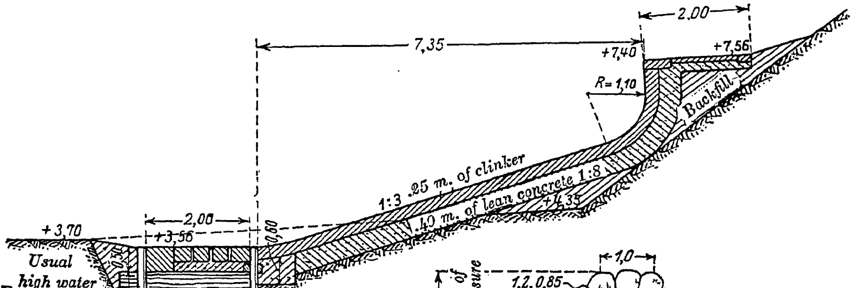


Fig. 144. Cover works at Wangeroog

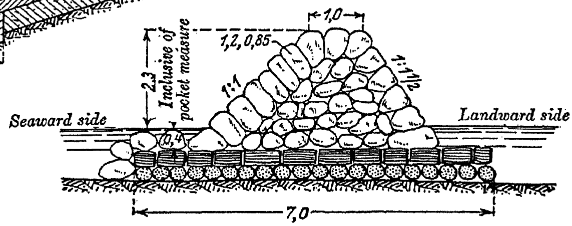


Fig. 145. Stone embankment on the Baltic Sea

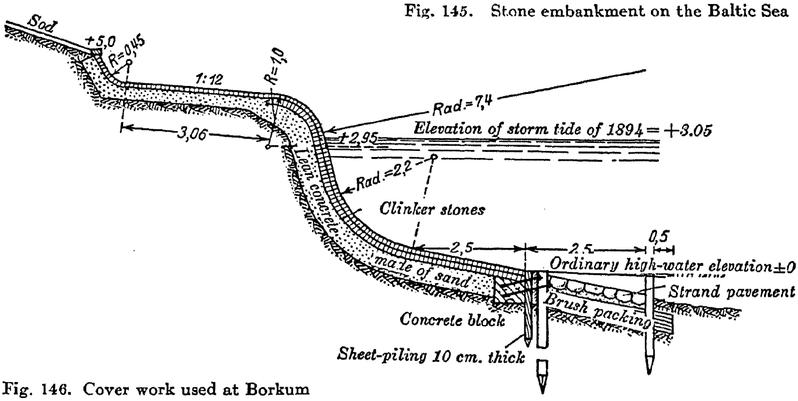


Fig. 146. Cover work used at Borkum

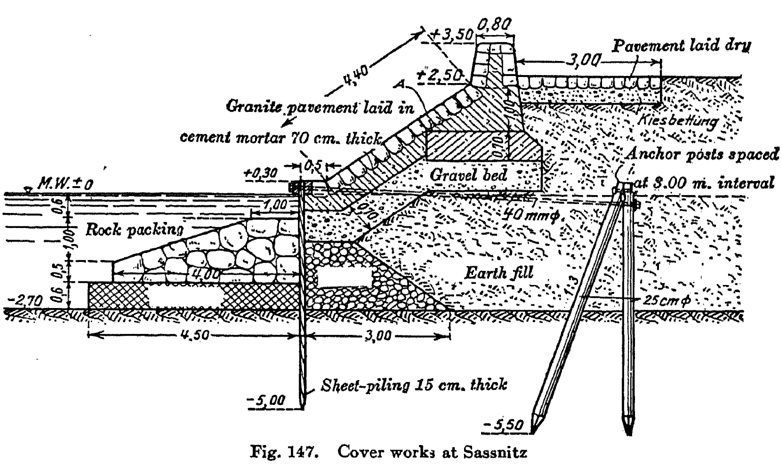


Fig. 147. Cover works at Sassnitz

forms c and d. This structure is built very high, extending 2 m. (6.6 ft.) over storm tide and is protected above by grass planted on the dunes. It should be noted that the intermediate stretch is built to a slope of 1 : 12. The wave peaks lose the largest portion of their energy on this part of the structure. This form of shore protection possesses many advantages but is very expensive.

A form which is more nearly like type d and b (Fig. 147) was used at Sassnitz. It has proven itself excellent for protection against strong wave attack. Sheet piling was first driven along the sea and anchored. Rubble fill was then placed in back of the piling in order that the back fill would not wash out. After this, wattlework loaded down with large stones was placed in front of the piling and the back was filled with rubble up to the water line. At this point operations ceased for the winter, a temporary cover of brush and rock being placed over the work. Thus, the part of the structure in place was well packed by the sea and then completed in the spring. A joint was left in the concrete at *A* by inserting cardboard during construction. In this structure the waves are dissipated at three points, at the foot, in the middle, and at the head.

c. Protruding Protection Works

Of late, much progress is being made in stabilizing shores by reinforced concrete structures. Extensive use is to be expected for this type of construction. However, it is necessary to contend with deterioration phenomena of concrete; the concrete, therefore, should be exceptionally well mixed, and only mineral cement should be used.[1] The results of work at Norderney are of interest in this regard. Various types of material were used there for fixing the heavy stone groins; these included heavy prisms of basalt rock, Portland cement, and mineral cement, which were all placed in the year 1908. In 1924 the mineral cement prisms were as good as new, having flat surfaces and sharp corners; on the other hand, the Portland cement prisms were partly disintegrated and the edges were rounded. The type of construction for strand groins is fundamentally different from that of river groins in shallow water, in that for the former, fascines and cover stones are fixed by piles which are driven into the strand. The effects of the two types also differ from each other. River groins are discussed in Part One of this book. It is assumed, therefore, that the nature and effect of river groins are familiar to the reader.

While river groins extend into deep water and must be safe from

[1] See Neue Uferbefestigungen in Eisenbeton. *Dt. Bauzg.* 1907, p. 18 of the appendix regarding reinforced concrete; *Dingl. Polyt. Journal,* 1907, p. 282; Engels, *Zentralbl. Bauverw.* 1911, p. 563.

current action, the strand groins, disregarding a few newer developments, usually lie flat on the strand and extend only slightly above its surface. They are expressly designed to provide resistance to the beating of waves. Consequently, they must have a sufficiently dense and heavy cover. They serve less to divert the coastal current than to create backwater in the fields between groins, under protection of which the sand settles. At usual water stages they are particularly effective in catching sand, but at storm tides they may temporarily act harmfully, large sand quantities often being washed out at that time. The combined action of strand groins, nevertheless, is invariably favorable to the strand. Of course, a preliminary stipulation for driving sand to the shore is the presence of sand in a coastal current or movable sand in the sea in front of the strand. It is also of importance if a deep channel develops in front of the groins because of tidal current. A deep channel of this type has developed at Norderney since some time ago. The sand-catching action of the groins is somewhat insufficient here. It appears that structures of the type now in use are valueless against such channel formations.

In localities where there is no regular coastal current, the length of each groin is not limited by a theoretical flank line as in rivers, but it is attempted to build every groin as far out as the low water allows in order to protect a maximum breadth of strand. The groin heads, therefore, lie near the edge of the low-water limit. Thus, they may be constructed in the dry, although in tide work.[1] Where a sufficient and regular coastal current exists, it is recommended to maintain also a flank in the sea.[2] The root of the groin must be safely carried into the shore or joined to the shore cover works. The root suffers severe attack, particularly under the influence of storm tides, and must, therefore, be extended up the shore over the highest water stage; or if the shore is stabilized by protective cover works, the groin must be well joined to the shore slope. The smaller the interval between groins, the greater will be their effectiveness. However, because of the high cost of groins (in Germany a groin of 200 m. (656 ft.) length may cost as much as 100,000 marks), they are spaced at greater intervals, about 100 to 200 m. (about 328 to 656 ft.) apart in order to protect as broad and long a strand field as possible. As time goes on, intermediate groins are installed.

As a result of overflow, wave attack, and return flow of water along

[1] Tide work is the name given to the very expensive work between HW stages; that is, during the short periods between about LW and MLW. It is also burdensome to the constructing engineer because of approximately one hour forward movement of the LW each day, making continual daily change in the working hours.

[2] Gerhardt, Die Befestigung d. Ostsee-küste bei Kranz; *Z. Bauw.*, 1906.

the groins at ebb, channels are readily developed along the edge of the structures. These channels gradually widen toward the center of the groin field. Their formation is combated by constructing spurs into the groin field normal to the principal groin, similar to fish bones. The intermediate groins must, therefore, be laid early enough to avoid complete lowering of the groin field. After these secondary groins have been placed, the elevation of the strand rises again. Where the shore is in the process of breaking down, the lowering of the strand will occur much more rapidly without the groins and will be permanent. In such instances it is best first to lay groins at a threefold interval of their length, the length extending from the fixed shore to the low-water line. At the root or shore end, the crest of the groin should extend at least 1 m. (3.28 ft.) above high water; the head should reach to spring low water, while the sides should be as flat as possible and laid on a 1 : 3 to 1 : 4 slope. It is not feasible to lay out a schematic system; the work must be varied to suit individual circumstances. A strand that has similar structures at equal intervals frequently indicates a schematic layout and thereby an offense against the spirit of genuine engineering. According to Dutch experiences, the groin backs should not project more than 60 cm. (23.6 in.) over the strand.

The strength of structure required depends principally upon the amount of wave attack. Accordingly, it is found that much weaker construction may be used in the Baltic than in the North Sea.

Examples: The simplest type of groins consists of single or double row pile walls. They may be used where there are no bore worms, but they do not have a long life if they extend too far above the strand, because waves and ice then impinge against them from one side or the other until they become loose. After being loosened, the water washes out individual piles, finally forming a gap, at the end of which a deep pool occurs.

The construction is considerably improved by spreading the rows of piles apart and laying stone between them, but the stones then gradually sink into the sand. The best form of pile groin is of the type shown in Fig. 148, the stone being laid upon fascine packwork. As long as the piles do not become dried out, such groins have unlimited permanence. Groins must be built differently in the North Sea because of the strong sea conditions there. As shown by high maintenance costs all of the lighter groins have proven unsatisfactory. Only the heaviest types of construction have proven successful along the coasts of the North Sea. Figs. 149 a to c

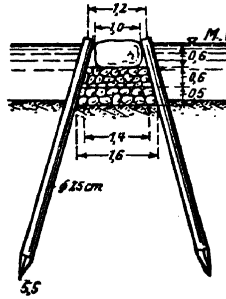


Fig. 148. Pile groin filled with rock superimposed upon fascines

give views of groins successfully used at Norderney and the other East Frisian Islands.

Fig. 150 shows a section of weaker intermediate groins, the resistance of which was not sufficient. These weaker works must often be widened to the full width of the groin indicated in Fig. 149 b. However, a groin which has been widened subsequently is not as strong as one constructed to its full breadth originally. A significant feature of these strand groins is the heavy stone cover, these stones being held in place by piles. Section a is taken directly in front of the root; b, near the head. From section a landward, the groin rises up to the height of the shore revetment. The top edge of the head lies .3 m. (.9 ft.) over ULW (usual low water), the lower edge some 1.2 m. (some 3.9 ft.) below ULW. The width of the groin proper increases from 5.8 m. (19.0 ft.) at the root to 8.7 m. (28.5 ft.) at the head. The blocks increase in thickness similarly from .3 to .5 m. (from .9 to 1.6 ft.). The piles vary according to the position of the strand, from 1.5 to 3 m. (4.9 to 9.8 ft.) in length, and from 9 to 18 cm. (3.5 to 7.1 in.) in thickness. The layer of broken rock is 15 to 20 cm. (5.9 to 7.9 in.) thick. At present, basalt prisms or concrete stones (mineral cement) are used for the surface layer. The groin head is made especially safe by fascines or rock fill. Brush-work fascines, constructed as in river control, are being used at Norderney. They are weighted down by basalt blocks. The use of gravel as an intermediate layer is unsuitable because of wave attack. The fascine bundles in this case were 50 to 60 cm. (19.7 to 23.6 in.) thick and 2.5 m. (8.2 ft.) long. In order to push the heads of these groins seaward as far as possible, they are constructed at LW of spring tide; hence, work can progress during only a few days at two-week intervals and during a very few hours of each day. The entire working force must, therefore, be concentrated for this work. Considerable skill must be exercised in properly laying out the procedure for such construction work. Recent experiments in the use of sheet piling for groins appear to be successful. Copper steel is especially suitable for this purpose.

A unique procedure was used in the maintenance of the dune island of Heligoland which lies at a distance of about 1,100 m. (3,600 ft.) east of the main island. The dune island, which is used principally as a bathing beach, was still connected with the main island up to the year 1720. The gypsum reef which tied the island to Heligoland was excavated for the gypsum to the extent that when the storm flood of December 31, 1720, occurred the dune was separated from the mainland and continually became smaller.[1]

The island lies in the center of a circle current which hindered

[1] A. Geisse, "Schutzbauten an der Helgoländer Düne," *Z. Bauw.*, 1905.

transportation of the sand into large depths. The problem of stabilizing the island consisted in finding a method to trap these sand masses. Other means were required than those of simply maintaining the sand. The solution was found in the use of groins; many of these extend to a

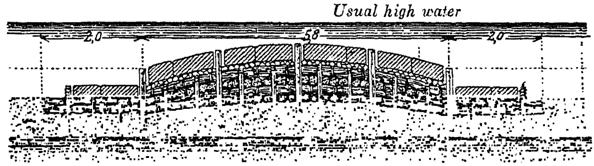


Fig. 149a. Main groin. Section through the foot

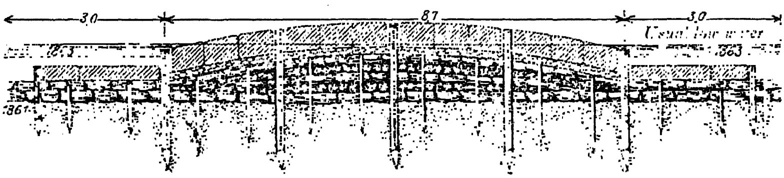


Fig. 149b. Main groin. Section through the head

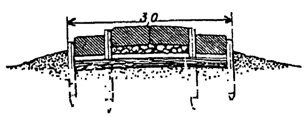


Fig. 150. Section through intermediate groin

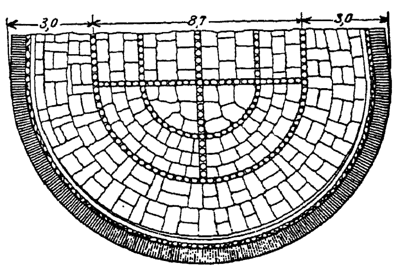


Fig. 149c. Plan view of the head of main groin

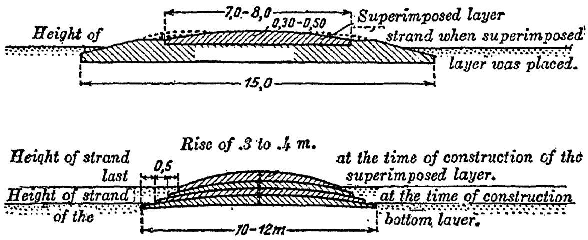
Figs. 149–150. Stone groins on the coast of the North Sea (Norderney)

bed depth of LW. They are installed to cause deposition of the moving sand so that it will be gradually pushed toward the strand by waves.

The groins extend radially from all sides of the dune island and reach a length of 900 m. (2,952 ft.). The part of the groin which lies above the LW elevation was constructed simply of fascines covered with sand. Under the LW elevation the groin was made of wattlework weighted down by rock. The wattlework mats consist of fascines tied together by wire and wick lines. It appeared unnecessary to weight down the mattress work by stone above LW because, after a short period, the groins were so completely covered with sand transported by the action of waves that they were never loosened by scour. Thus it

became possible to raise these upper portions of the groins readily by renewed mattress layers (Fig. 151 b).

Fig. 151 a shows a section through a piece of wattlework in the deeper portion of the groin. The strand was originally at the same elevation as the lower side of the wattlework. The effectiveness of the groins is recognizable in both figures. The construction of the groins has been somewhat extensively explained because this is an experiment of special significance.



Figs. 151 a and b. Cross-section through groins at Heligoland

To the present time the maintenance of this dune island has been assured through the construction of the groins. The work would probably not have been successful had they adopted the original plan of protecting the strand with cover works. The expectations fostered by the designer of this project, the late Director Franzius, Bremen, have not been entirely fulfilled. The dunes are simply not decreasing in size, but even this is a success of the first order.

C. MARSHES AND LEVEES

a. The Arising and Cultivation of Marshes[1]

Silt is mentioned in Part One as the medium through which new land is formed on the coasts. Material from the newer formations of this land, called marsh, is required in harbor structures when it is first necessary to surround the site by dikes. A knowledge of the properties of marshland is of importance in the construction of harbors because they frequently lie in marsh districts. Marshes are classed among the most recent geologic formations (the alluvium), and are recognized as the most fruitful agricultural districts. Young, newly endiked marshes produce twofold harvests. In spite of catastrophes of the first order, inhabitants of the fertile lowlands invariably return to their soils, conquering the overflowed area. This is particularly true of the Dutch and Frisian marsh inhabitants. Natural marshes are found in many places on the earth, especially in India, Egypt, China, Texas, Florida, and South America. Many river marshes are as fruitful as sea marshes. The most important marshes of Europe are those along the North Sea coast.

[1] *Prometheus*, 1912, p. 231. Woebcken, *Deiche u. Sturmfluten*, Friesen, publisher, 1924.

Here new land is frequently reclaimed. The inhabitants have seldom wandered away from these rich lands; when they did leave, it was because of the absolute impossibility of reclaiming the lost land (Cimbri and Teutons, Frisians, etc.). They have often worked for a period of a half generation (once for eighteen years) to regain the land, or until the attempt finally had to be given up. Usually, however, the reclamation of inundated land has been successful. The history of the German marshlands is one of the most interesting of the German people. The inhabitants of the land behind the sea dikes were continually in danger of losing their lives. There is little doubt that in the course of history hundreds of thousands of men, women, and children have become the prey of storm tides.

New land may form where there is sufficient fine, alluvial deposit, provided the coastal or tidal currents are not strong enough to carry away the deposits which form during the quiet period at changing tide. New marshes form principally in bays and in back of islands; in exceptional cases, development may also occur in the sea in front of the open coast. The fine sediment falls to the ground only where the water is free from waves and current for long periods of time. The help of sea plants, which cause a practically complete quieting of the water, is usually necessary. The growth of plants is first fostered by rich silt. but eventually enough silt falls to smother the plants. A new plant layer forms which gradually becomes covered by the fine deposit. Layers of peaty plants are found in many marshes, often in numerous superimposed layers. Each layer may be several meters thick. In the reclamation of marshlands, it is important to know whether the marsh lies directly on the sand or on peaty plant layers since the latter shrink very much more than the former as the result of rotting.

The shore land developed by the falling of silt is called tidal land. As long as tidal land lies below low-water, it grows rapidly in height, but when it is inundated only during HW stages the rate of rise is slower. The part lying higher than the usual high-water develops extremely slowly.

Tidal land begins to develop vegetation as soon as the surface lies at an elevation above the mean tidal rise. Various salt weeds begin to grow at this stage of development. The growth gradually becomes more luxuriant under the continual influence of the sun and wind.

When the tidal land has become traversible, the marsh formation is fostered artificially. Ditches are made and low levees are constructed of the excavated material, thus holding the water which flows over the shore during HW long enough to cause a maximum amount of silt to deposit. The higher the tidal land rises, the more luxuriant the plant

life becomes. However, grass does not begin to grow until the marsh reaches a height corresponding to the elevation of the usual high water. Tidal land is usually traversed by stream channels in which the water may flow back into the sea during ebb. These channels make it difficult to surround the delta with levees.

The period from the time of traversibility of a marsh to the time when the surface of the marsh reaches the elevation of ordinary high water usually amounts to about twenty-five years, and a further twenty-five years up to the time when the elevation will have been raised .5 m. (1.6 ft.) more.

Without artificial cultivation, tidal land would require about one hundred years to reach the same stage of development otherwise requiring fifty years. After this period of development, the land eventually becomes ready to be surrounded by levees, thereby converting it into polder land.

The period during which the tidal land is cultivated into polders is a very important stage of development. All marshes sink as a result of drainage. The more water that is taken from the clay soil while drying, the less will be the amount of subsequent sinking. This is still more pronounced in connection with the strata composed of sea plants. Many East Frisian and Dutch marshes have sunk as much as 2 m. (7 ft.) due to shrinkage upon drying. The sinking of the entire Dutch

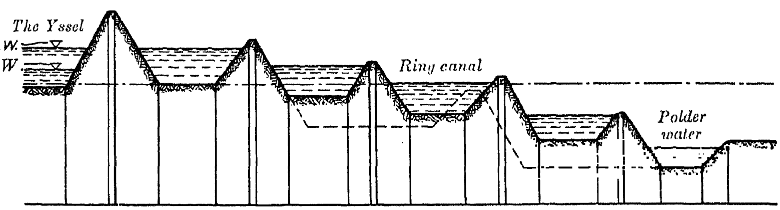


Fig. 152. Section through the Zuidplas Polder

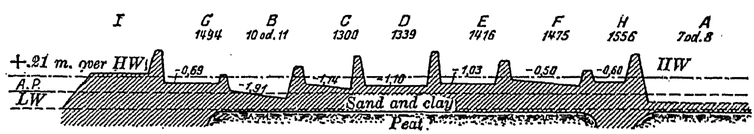


Fig. 153. Elevations of the polder at Tholen

and East Frisian coast because of recent geologic changes is insignificant compared to that resulting from shrinkage. The former has averaged about 1.8 mm. per year in recent years, thus probably amounting to 36 cm. (14.2 in.) within the last two hundred years.[1] The rate of lowering may have been greater formerly.

[1] *Annalen der Hydrographie*, 1909, p. 81, and Martin, "Beitrag zur säkularen Senkung der Nordseeküste," Vol. 17 of the *Jahrbuch f. Altertumskunde.*

Since the new polder lies seaward from the old, very unfavorable drainage conditions exist for the old, because the new foreland invariably lies much higher than the old polder (Figs. 152 and 153). In order to reduce the amount of sinking to a minimum, polders should be formed only after the tidal land has reached an elevation at which there is practically no additional rise resulting from the deposition of silt. This condition obtains when the tidal land lies .3 to .5 m. (1 to 1.6 ft.) over usual high water.

In earlier centuries, polders have been developed too early. Although fruitful land was won from the sea at an earlier date, the apparent gain later turned into losses which can never be made good. Marshes which shrink and soon lie too deep, require more expensive drainage arrangements and are more severely threatened by storm tides, because of the greater strain upon the levees and greater depth of inundation in case a levee breaks.

b. Levees

1. Purpose and Classification of Levees

Levees (also called dikes) are earth embankments constructed on the coasts for protection against high tides of the sea, and on river shores for protection against inundation of the lowlands. Presumably the first sea dikes in continental Europe were constructed about 1000 A.D. Before this it was customary in the Netherlands, probably for one thousand years, to construct wharfs. The wharfs were artificial earth mounds, usually of such giant magnitude that they carried entire communities. Before the construction of levees these mounds were surrounded by water during extreme high tides, so that they protruded upward as islands. They were constructed in three different layers. The wharfs of North Friesland usually accommodated individual courts or small groups of houses. Wharfs are built to the present day on the small low islands. Clay is excavated from deep ditches and compacted into the wharf in layers of 50 cm. (20 in.) thickness up to heights of 4 to 5 m. (13 to 16 ft.). The ditch from which the material is excavated completely fills with silt within ten to twenty years. The large wharfs were also probably constructed in this way, the sea repeatedly carrying new building material to the site. But since the storm tides continually become more frequent, stronger, and more dangerous, probably because of sinking of the coast, it was resolved to construct levees for the protection of East Friesland. Thus, the "Golden Ring" was built. Levee construction is one of the greatest deeds of past generations. As a result of this undertaking, great portions of the North Sea coast have been saved from submergence, and large new stretches of land have

been reclaimed. River levees were probably constructed along the Rhine long before the construction of sea levees, and levees were possibly erected still earlier by the great ancient races.

In the construction of sea harbors, because of various circumstances, levees must be moved frequently or built anew. If a harbor is to protrude far into a polder, the land must be surrounded by levees beforehand; frequently the existing levees must be pierced and the resulting gap closed by a sea lock. Similarly, river levees must often be built for the purpose of regulation or canalization.

Levees are divided into two large groups, sea levees and river levees. In addition to these, another group may be segregated; namely, canal levees. The latter are usually designated as canal embankments, although they are actually levees.

There is no sharp difference, however, between river and sea levees, since sea levees are built inland along the river mouth as far as the influence of the sea extends. From the point where the river endangers adjacent territory more than the high-water of the sea, the dikes are classified as river levees. Of course, there is a transition section along which both river and sea influences are of importance. The presence of ebb and tide or salt and fresh water in front of the levees is not a deciding factor with reference to the classifications. For example, in the city of Bremen, tides of 2 m. (6.56 ft.) height are felt although only fresh water is present in this part of the Weser River. Along the Baltic Sea coast the water in front of the levees is pure sea water, but there are no tidal changes.

River levees are constructed for the purpose of protecting the lowlands and bettering the river. They hold together the high-water flow of the river and thereby improve the conditions of detritus movement by hindering irregular deposition. In many cases the levees of large streams were laid originally without any consideration of the problems of river control. The alternating broad and narrow intervals between many of these older levees have serious effects during high-water stages. Such levees are attacked principally by the current and ice, and less by waves. River levees need extend but little [.3 to .5 m. (1 to 1.6 ft.)] above the highest high-water.

Sea levees are not designed to hold currents together; their position, even in tidal funnels of rivers, is unimportant, since there is not a significant amount of narrowing. Sea levees are seldom threatened by currents, waves being the principal source of danger. During storm tides, currents are negligible compared to the impact of waves. These dikes must, therefore, be substantially higher above the highest high-water than river levees. The highest possible wave which is likely to strike the dike must not reach the top. Destruction of dikes on sea fronts in the past has been caused almost entirely by the beating of the waves or overtopping by high-water, not by floating ice or overtopping

because of ice blockades. Saturation with water is also a danger for sea levees, because serious high-water stages, even in tidal districts, may sink so little during long storm periods that large portions of the embankment may remain under water continually and be attacked by waves at the same time. On the Baltic Sea coast, the high-water may remain unchanged for several days. Saturation of sea levees, however, is not as dangerous as that of river levees, because the high-water at the sea lasts only for days, while in the case of rivers it may last for a period of weeks.

The saturation of all levees plays an important part in their stability, the line of saturation being a deciding factor for the dimensions of the structure. Further consideration is given this factor in the investigation of types of levees.

Sea dikes as well as levees along rivers are described as either main levees or secondary levees. At the sea, the main levees are often called winter dikes because the principal danger is expected there in winter, but this designation is also used for levees along rivers. Corresponding to this, the designation summer levee is used for secondary dikes which, at the sea and along rivers, hold the low summer high water away from the foreland. The summer levees lie in front of the main or winter levees. The winter levee is also designated as *ban* or inspection dike because it is under ban or dike inspection (by the dike captain or dike magistrate in Germany). There has been a ban on main embankments since ancient times with a heavy penalty for anyone who damages the dike. The secondary levees are arranged to hold the silt, which is carried over into the polder at high-water stages, between the main dike and secondary dike, and thereby raise the land and make it more fruitful.

Ban dikes have little protective foreland, so that the sea motion, or the current in the case of rivers, strikes them with full force; they are

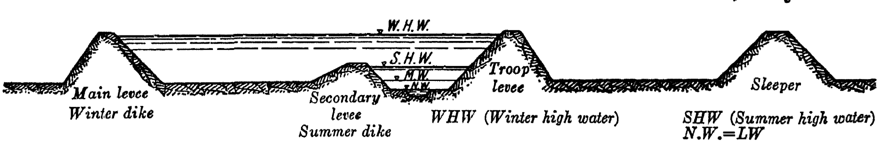


Fig. 154. River levees

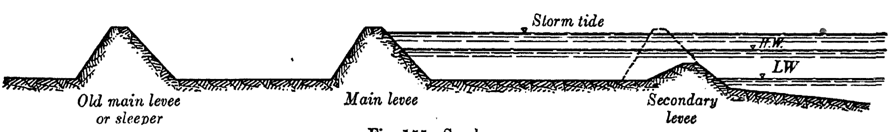


Fig. 155. Sea levees

danger levees. The threatening danger of a break in the levee must often be counteracted by the construction of an intermediate dike.

Spare dikes or "sleepers," in contrast, are old main levees which have been replaced through the construction of new main levees in

front of the older structures, thus creating new polder land. When a levee has become a ban dike, then, of course, a spare dike lying further from the water is very valuable. These occur mostly on the seashore. The various types of levees are illustrated for rivers and for the sea in Figs. 154 and 155. The levees are drawn to a distorted scale.

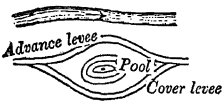


Fig. 156. Advance levee

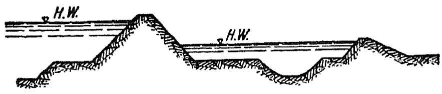


Fig. 157. Cover levee

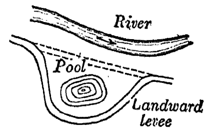


Fig. 158. Levee landward of pool

In addition to main levees there are also wing dikes which serve the purpose of protecting especially endangered stretches of levee from the sharp attack of water or ice. Strictly speaking, they do not earn the name of dike because they serve as protection dams, wave breakers, or guide dams but not as dikes. (The latter term, however, is frequently used and has become the vernacular in the Mississippi Valley.) They must usually be constructed with especially heavy slope protection (stone paving), but a very flat form of slope is not so necessary as for main levees.

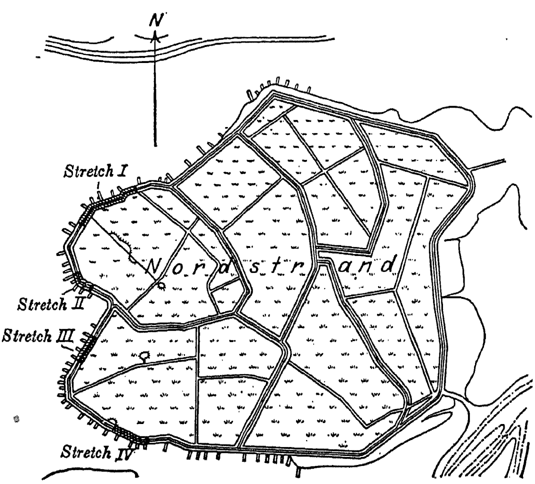


Fig. 159. Ring levee at Nordstrand

Ring levees are of closed ring-shaped plan, and are used for the protection of individual communities (Fig. 159). The water often soaking through a dike results in springs or "sand boils." Where these springs occur to a dangerous extent, curve dams are constructed behind the dike (Figs. 157 and 158) and act as intermediate levees for the protection of the lowlands. At the sea there is also the storm levee which lies back of an endangered main levee as added protection, and takes the place of the main levee in case of a break.

The slope-stabilization material for levees may be sod, straw, brush, or rock; the building material for the levee proper, clay or sand.[1]

[1] Grässner, *Zeitschr. d. Bauk.* 1911, p. 565; *Prometheus*, 1912, p. 232. *Denkschrift zur Vollendung der Bedeichung der Bredeau-Niederung*, by dike inspector Hinrichs.

2. Position, Cross-Section, and Height of Sea Levees

The cross-section and the height over the highest high water are dependent upon the form of the shore to be protected and the position of the levee with reference to the most dangerous storm direction. The more nearly the levees are normal to the storm and the direction of the wave attack, the more they are endangered. Hence special consideration must be given the form of line at which a new levee is to be laid, either for enclosing polders or displacing levees for harbor structures. If at all feasible, new levees should be so laid that the principal direction of attack is met by a mild curve. Sharp corners in alignment should be avoided. In the interest of diminishing the cost, the shortest line is always the best as long as the levee remains safe. Dikes should preferably follow the course of channels. Special precautionary measures are necessary in this connection.[1]

Where no broad tidal land has formed in front of the shore, a sufficiently wide foreland must remain protected in front of the levee. This moderates the waves and, as in the case of river levees, is necessary for a source of earth for maintaining the embankments. The foreland should be at least 120 m. (394 ft.) wide; in places where wave attack is particularly strong, 350 m. (1,150 ft.). The figure is dependent upon local circumstances. The stretch from the foot of the dike to the low-water shore is considered foreland. Maintenance of the foreland is usually as important as the maintenance of the levees themselves, even more so than in rivers. If shore protection works are too scant, the foreland is frequently washed away and the levee thereby transformed into a troop dike whose maintenance and defense becomes more costly than the former protection work and levee combined.

In forming polders the levee must lie back far enough so that the immature flood land remains a considerable distance from the embankment. Otherwise, in place of natural drainage, a very much more expensive system will be necessary. In closing off bays, such as the Zuider Zee, the formation of polders in sandy districts is avoided as far as possible because in such regions sand boils are continually encountered. Summer levees should be constructed only on foreland already vegetated.

The top of the levee must lie higher than the wave crest of the highest known storm tide. The height of this tide depends upon the form of the coast and is greater in pointed bays than along straight shores. In Germany the height of the storm tide of 1825 and in north Holland that of 1872 is used as a controlling elevation. The tide of February 3 and 4, 1825, along the German coast reached a height of 3 to 4 m. (7 to

[1] Formerly diking over channels was not risked.

13 ft.) above the usual tide. The tide of 1872 overtopped the elevation of the tide along the Dutch coast of 1825 by 60 cm. (23.6 in.). The storm tide of February 16 and 17, 1916, in some places was even greater than the previous one. Geologic changes in the sea bottom and coasts may bring substantial changes in the tide heights. The wave height increases the more nearly the levee lies normal to the direction of approach, the steeper the outer slope of the levee, and the narrower the foreland. The greatest wave height in front of levees on the open sea without foreland may amount to 3 m. (10 ft.) along the German and Dutch North Sea coast, 4.5 m. (14.8 ft.) along the English sea coast, but the height amounts to only .3 m. (1 ft.) in rivers at their tidal limit. The height of the crest of the levee over highest storm tide should, therefore, in Germany, be not more than 3.3 m. (10.8 ft.) but at least .6 m. (2 ft.) at the stretches not endangered. Summer levees are seldom built and those built are often later transformed into main levees. Their crests of about 2 m. (about 7 ft.) width lie 2 to 3 m. (7 to 10 ft.) over ordinary tide.

Corresponding to these viewpoints, raising the levee crown for long stretches results in a wave line, the crest of which lies at the most endangered locations. *Experience with the great storm tides of the past shows that complete safety of a levee should never be expected; furthermore, there is no basis for believing that future tides cannot exceed the heights of any previous tides.*

TABLE NO. 2

SUMMARY OF HEIGHTS OF SEVERAL NORTH SEA LEVEES

(LISTED FROM THE EAST TOWARD THE WEST)

Location of the Coast	Height of Crown Over			
	Storm Tide		Ordinary Tide	
	m.	ft.	m.	ft.
Holtstein west coast (Dithmarshes)	1.8 to 2.8	5.9 to 9.2	5.3 to 6.3	17.4 to 20.7
Weser levee	0.6 to 1.2	1.9 to 3.9	4 to 4.6	13.1 to 15.1
Between Weser and Jade	1.8	5.9	5.0	16.4
Jade levee	1.60 to 2.20	5.2 to 7.21	5.3 to 5.8	17.4 to 19
East Frisian levee	1.7 to 2.2	5.6 to 7.2	5.2 to 5.7	17.1 to 18.7
Raised Groning levee (In Holland)	....	....	5.8	17.4
Zuider Zee	....	....	3.5 to 3.8	11.5 to 12.5
North Sea levee at Helder	....	....	3.7 to 4.8	12.1 to 15.7
West Kapel levee on the Schelde	3.3	10.8	5.55	18.2

In Germany the width of the crown averages 3 to 4 m. (10 to 13 ft.); in Holland, as much as 8 m. (26 ft.). In Germany the inner berm is

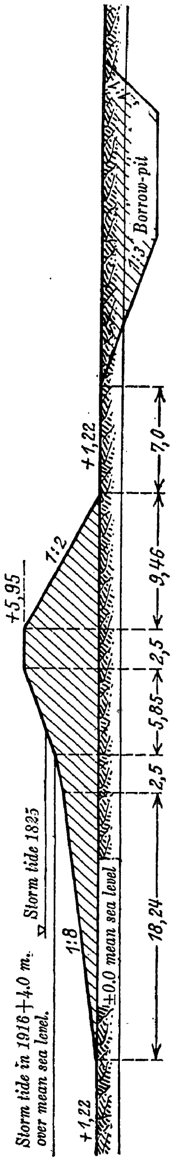


Fig. 160. Levee cross-section proposed by Hinrichs

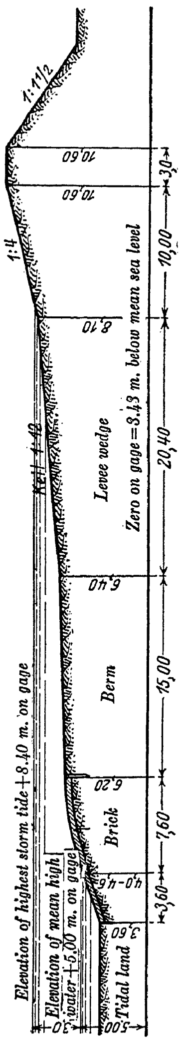


Fig. 161. Levee on the Jadebusen—Jade Bay)

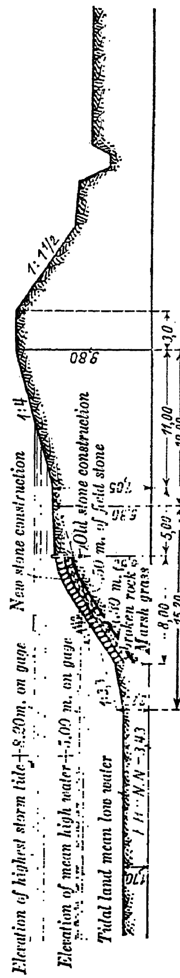


Fig. 162. Troop levee on the Ahne (Jadebusen)

used as a roadway, in Holland the crown is generally used as a transportation way. With similar side slopes, the wide levee is the safest.

The stronger the beating of the waves, the flatter the outer slope must be. At the most endangered places directly on the sea, the side slopes have been constructed at a slope of 1:12; in river mouths, as steep as 2:3.

Instead of a straight outer slope, many levees have an outward or concave slope; the latter will probably prove more advantageous. (See Figs. 160 and 161.) Fig. 162 is an example of a levee having a convex outer slope. The outer berm is of great importance. It is especially effective in dampening waves, and many old levees have had this added subsequently to the original construction. The inclination of the inner slope amounts to 1:1.5 to 1:2.

All of these cross-sections should have a particularly favorable influence on the permanence of the levee. It is very important to provide a protective covering for the outer slope and good sodding for the crown and inner slope. No piles should be driven in the outer slope of the levee. They become loosened by the waves to the extent that crevices are formed where destructive action may begin.[1] The line of saturation is of particular influence in connection with the design of cross-sections. The outer slope should be made flat enough so that the line of saturation remains within the body of the dam. This line begins at the point of intersection of the dam with the highest water stage, and has an inclination of about 1:6 to 1:8, depending largely upon the nature of the material.

3. Position, Cross-Section, and Height of River Levees

In the arrangement of river levees, the favorable choice of land is not the only factor of importance; it is also important to lay out a plan so that the HW channel of the river is not too greatly narrowed. In addition to the danger to newly developed stretches, serious narrowing will cause an immediate danger to the stretches further upstream. The necessary widths of section of the river channel must be chosen with care and checked by computations, so that the HHW stage will not rise above a definite limit. Computations for the backwater curves for HHW will indicate the effect to be expected as a result of the construction of levees. Levees are made as slender as possible, special consideration being given to the location of the axis of the stream so that serious attack on the levees, particularly by ice, is avoided. It is obvious that ice blockades will take place most rapidly and readily at abruptly narrowed sections. All transitions of the levee line must be gradual. The local position of the levee is important, since sufficient

[1] In the storm of 1916, the Cuxhaven levee was practically broken as a result of such piles.

foreland must remain to furnish material for the embankment. Often deviation is made from the ideal levee line and the levees set further back from the river in order to avoid saturation of unreliable lands. This displacement is not uneconomical when the ground is cheap and not usable for agriculture. If the foreland is fruitful, alluvial ground, it is often uneconomical to move the levees back from the river.

The same factors come into consideration in designing the cross-section of river levees as in sea levees. Summer dikes form an exception when they are to serve as overflow embankments. The main levees must have flat slopes on the river side, the slopes being about 1:3 to 1:5. Frequently the material used is also an influential factor in this connection. Fig. 154 shows a view of a winter levee used on large rivers. Generally the slope of the back side of a levee may be made between 1:1.5 and 1:2. The line of saturation is a controlling factor, yet it is important to recognize that a ditch laid at the inner foot may become dangerous in case of strong penetration of the levee. The foot of the inner slope may be made safe by the use of gravel fill.

Fig. 163 indicates a levee section for small rivers. The same section may also be used for summer dikes. The width of the crown is so chosen

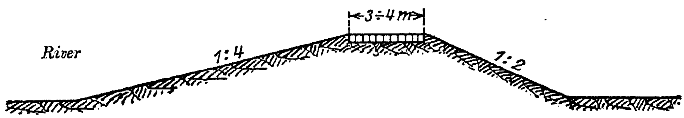


Fig. 163. Levee for small rivers having crown breadth of 3 to 4 m.

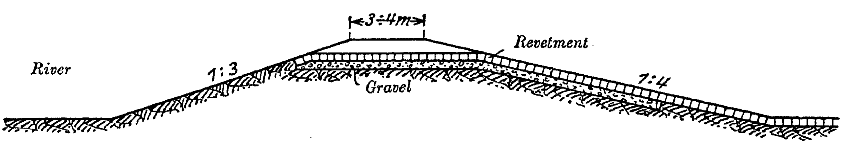


Fig. 164. Overflow in a smaller levee

that it may be used for a roadway, usually 3 to 4 m. (10 to 13 ft.) will be sufficient. If a summer dike is to be overflowed, the section is reversed. Then, according to Fig. 164, the inner slope must be flat so as not to be destroyed by overflow. Usually it is preferable to provide a particular overflow stretch for a summer dike, the length being such as to allow rapid enough overflow to fill the space behind the summer dike before general overflow over the entire levee crown takes place. This particular overflow stretch must lie .3 to .5 m. (1 to 1.6 ft.) lower than the crown of the levee. The inner slope of this stretch should preferably be paved.

4. Material and Construction of Levees

In locating levees, unreliable ground should be avoided where possible. This, however, is practicable only when large deviations are not necessary. It is generally possible to choose solid foundations for river levees more readily than for sea levees. Along the sea and at bad places along rivers, levees must often be continually built up and allowed to sink until a firm foundation is attained, which often results in a significant increase in the necessary quantity of material. Setting of ocean dikes is a particularly expensive process. Here in particularly unfavorable cases, the amount of material needed may amount to as much as 200 per cent of what is necessary for a normal stretch of levee. It sometimes occurs that levees sink into the soft substrata to the extent that they disappear entirely. All levees continue to settle for several years after construction; therefore, sufficient excess height should be allowed during construction.

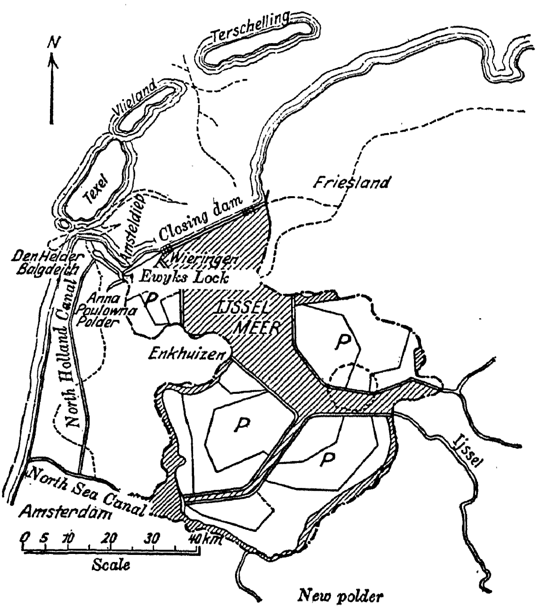


Fig. 165. Closure of the Zuider Zee for the purpose of forming new polder land

Before building up an embankment, the site should be cleaned of all roots, wood, and the like, so that a good junction between it and the new levee may be obtained. Preferably the ground should also be plowed over before earth is superimposed. This preliminary treatment reduces the subsequent penetration of water. If it appears that penetration will continue to threaten the structure, then, especially for rivers with sandy substrata, a clay core should extend downward to a stratum not easily penetrated by water.

The best material for levees is clay or loamy soil. Great stress should be laid upon the purity of the earth used in levees. All roots, plant remains, wood, etc., should be removed so that no hollow spaces will develop subsequently through decay within the embankments. In

using the best levee earth, clay, all dead plant life must be carefully removed. Levees being constructed in Holland for the purpose of reclaiming land from the Zuider Zee[1] are made of a material called "keileem," a type of clay dredged locally which has an extraordinary resistance against wave attack. Fig. 166 shows a section of the new Zuider Zee dikes.

If no clay is available, a mixture of clay and sand, or still better, a natural loam (which is a mixture of clay and sand) may be used. Pure

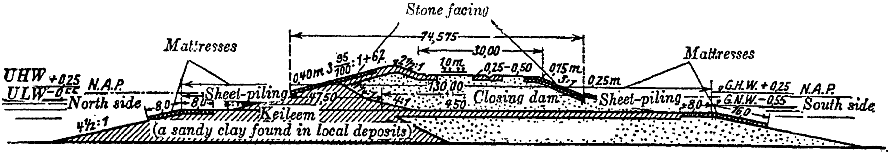


Fig. 166. Closing dam of the Zuider Zee

sand must either be given a clay core or a safe impermeable outer cover. A clay cover is preferable in the latter case.

An example of a clay covered dike is given in Fig. 167. The thickness of the clay layer on the outer berm slope, upper slope, crown, and inner slope, respectively, is .5 m., .4 m., .2 m., and .2 m. (1.6 ft., 1.3 ft., .7 ft., and .7 ft.). Protection at the outside under high-water is provided by straw stitching, an inferior cover which is no longer used in better designs.

The embankment material should be placed in thin layers of 30 to 40 cm. (12 to 16 in.) thickness, having an inclination toward the water side so that rain water may flow off. Formerly the earth was excellently packed by horses walking over the fill; at present, power rollers are used to pack the material. This method has proven itself particularly

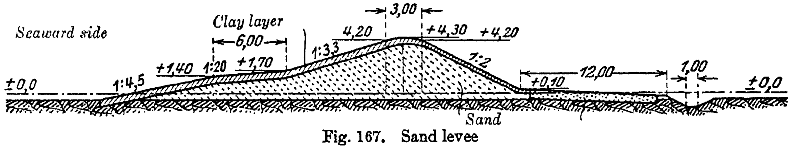


Fig. 167. Sand levee

satisfactory for canal embankments and may be used with equal success for levees. Stamping by hand is of uncertain effectiveness. Because of the compression of the ground by stamping, a corresponding addition of 20 to 25 per cent of material must be used and the embankment must be made 8 to 10 per cent higher; for soft underground, as much as 25 per cent higher, in order that the correct height be retained after settlement takes place.

[1] The Zuider Zee (Fig. 165) was formed by the storm tides in the thirteenth century.

The destruction of sea levees is due not only to scour resulting from the beating of waves on the outer slope, but also to the action of waves overtopping the structure and attacking the inner slope. Protective cover must, therefore, also be provided on the inner slope.

Below ordinary tides, the slope must be protected by a particularly good covering. Straw stitching is frequently used on the lower part of the outer slope; this consists of a layer of straw 2 to 3 cm. (0.788 to 1.182 in.) thick which is fixed to the earth by straw cord in so-called straw stitching. Rye straw is best for this covering. The method is usable only on shale ground. Other protective surfaces consist of brush covering having a sublayer of straw, willow matting with sea grass, and in emergency, canvas, all of which are fixed to the slope by stakes. All of these types must be renewed frequently, in some cases every year, so that in the end they are just as expensive as permanent protection works and do not provide the same degree of safety. Renewals are usually made in the autumn.

In localities where the foreland is high, extending above normal tide, the entire levee is usually protected by means of a grass cover. A good grass cover is obtainable either by laying sod or by sowing seed. Sod must be laid as fresh as possible, because otherwise growth becomes difficult. The sod pieces should have rectangular edges. After being laid, the sod is well tamped and then strewn with humus earth. If seed is used, it must be planted relatively earlier than sod; a particularly qualified grass seed must be chosen and sowed on a 30 cm. (12 in.) layer of good ground. Sowing of sea dikes is a particular art, because salt grasses must be chosen for them. The work may be trusted only to experienced gardeners who understand the choosing of the correct combination of grasses as well as sowing and nursing them. There are few such gardeners, but they may be found along the coast.

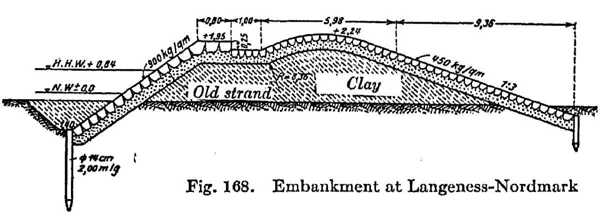


Fig. 168. Embankment at Langeness-Nordmark

The best permanent cover consists of stone paving. It is constructed like that used for shore cover works. Natural quarry stones weighing from 100 to 500 km. (220 to 1,100 lb.) are placed on gravel. These may be of granite, basalt, hard sandstone, or concrete blocks. In sand levees, clay or marsh ground is used as a substratum under the stone paving. In Oldenburg and Holland, clinker is successfully used for

protective covering. When stone covers are used, the outer slope is made comparatively steep, it being cheaper to widen the berm somewhat and provide a steep slope in front of it than to pave a very wide flat slope (Figs. 161 and 162). The paving of river levees, or levees at dangerous locations along the sea coast, should invariably be laid on a cover of gravel or crushed rock. This sublayer serves to protect the earth embankment against scour through the joints of the paving. The paving is 30 to 40 cm. (12 to 16 in.) thick; the gravel layer, some 25 or 30 cm. (10 to 12 in.) thick. Fig. 168 shows the dam at Langeness-Nordmark at the point of greatest strength. At locations having less attack to withstand, the crown (elevation+2.24) and the inner slope (1: 3) are protected by thick grass sod which, however, does not withstand the attack of overtopping waves. The form indicated in the figure is the safest; however, the arrangement of the water bolster of 1 m. (3.28 ft.) width is of little value. Hansen[1] intimates that it is better immediately behind the division at +1.95 m. (+6.39 ft.) elevation to continue with an inclination to the crown (elevation+2.24). This view also agrees with experiments made by Krey on the weak effect of flat water bolsters. The outer slope is very steep and has a stone cover of 900 km. per sq. m. (1,650 lb. per sq. yd.); the slope is approximately 1: 1.25.

The covering of river levees need not be as heavy as that of sea dikes, but it is desirable not to skimp in either case. Most levees have been planted with grass. Straw stitching and bush cover are not used. Stone has been used to build river levees, but seldom as compared to sea dikes. Levees with stone cover are used particularly along mountain streams.

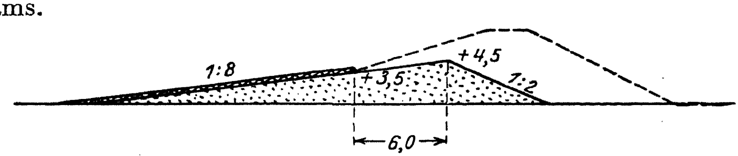


Fig. 169. Levee construction proposed by Hinrichs

If practicable, levee construction should be finished by the end of September, so that the earth can settle and the growth of grass be assured. If entire completion of the levee is impossible because of the great length, the levee (as indicated in Fig. 169) may be first built to approximately half its final height and protected for the winter by temporary covering, or a part of the levee may be entirely completed and the ends protected against the attack of the waves and currents. Levees

[1] Hansen, doctorate dissertation, Untersuchung von parallelen Uferschutzwerken, Technical University at Hanover, 1924, Fig. 19.

are frequently constructed to their entire length at half the final height, because with exclusion of high floods the purpose of the levee is often already attained with the structure at half height. This method is very expensive, however, and not always applicable.

5. Maintenance and Protection of Levees

In many regions protected by levees, the settlers have established so-called levee syndicates, each of which is under the control of a levee captain or levee magistrate. All members of the organization must aid in maintaining the levee. The maintenance of the levees consists in repairing all damages promptly, holes made by all rats, moles, rabbits, and the like being closed as soon as discovered, because such holes frequently are the starting points for the destruction of sea levees during storm tides. Bushes or trees should be hindered from growing on the levee slope. Holes arising at high-water should be filled immediately, and damages to the crown of the embankment caused by wagon tracks should also be repaired. In case the levee settles too greatly it should be built up.

The foreland beyond the levee should be particularly observed. The permanence of the embankment depends upon the maintenance of this stretch of land. If the foreland recedes because of the current or wave attack, measures must be taken to hold it intact by constructing a series of groins or a protective covering; however, it should not be taken for granted that these precautions always overcome the difficulty.

During hours of particular danger, the dike magistrate has a right to demand the help of all members of the syndicate. Defense of the levee is recognized as an affair of all adjacent inhabitants. Each one has the duty of placing all available materials[1] and all working forces at the disposal of the magistrate. As soon as the high water has reached a dangerous stage, for example, at half the height of the dike, regular sentries must traverse the dike, and materials for various necessary repairs of the dike must be supplied. Formerly there were no sentries and, in consequence, inhabitants were sometimes overtaken by the sea while they were asleep.

No special means of help are available against the extraordinary heavy sea; but, in the case of rivers, it is often helpful to lay fascine mats on the outer slope of the levee. These mats should be so placed that they reach at least 1 to 1.5 m. below the water level. Old mattresses, canvas, and the like may also be used in case of necessity. Leaks should be stopped by means of sand bags, a large quantity of which should always be at hand. Frequently the point of entrance of the

[1] Even beds, furniture, house doors, pianos, and the like can be demanded.

water can not be found, so that covering with sand bags is not possible. In such cases the exit of the spring on the inside must be protected by a cofferdam constructed of sand bags or piling (Figs. 170 and 171). This

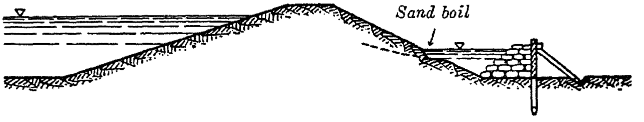


Fig. 170

Fig. 171

Figs. 170 and 171. Method of stopping a sand boil

construction must be rapidly strengthened and made as impermeable as possible so as to build up water pressure at the exit which will provide resistance to further flow. Even though the water level on the inside of the cofferdam is not held as high as it stands on the river side, the movement of water is slowed up considerably so that the hole in the levee may become stopped up of its own accord. A tested divining rod should be available for hunting such springs. Many people possess this faculty, but their reliability must be tested beforehand. At the sea during heaviest storm tides, waves may beat over the crown and thus endanger the back slope. As soon as the crown starts caving, it must be repaired with great speed since progress of destruction from the back out may progress very rapidly. Reinforcement, by means of a cofferdam constructed on the water side, is necessary when the back slope starts caving (Fig.

Fig. 172

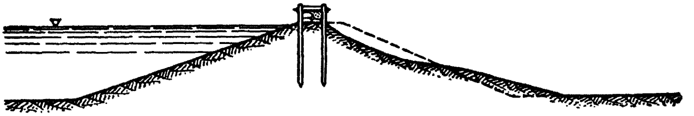


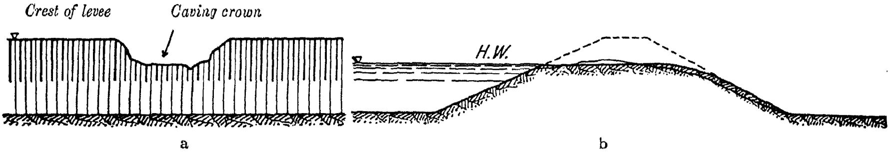
Fig. 173

Figs. 172 and 173. Protection against slides

172). In the case of rivers, the feasibility of raising the levee may come into question. The ice sentry service will give news in due time concerning whether an ice jam threatens. In case a jam forms, the water

may rise extremely fast. Sometimes relief may be obtained by laying up several lines of sand bags. The construction of a low cofferdam on the crown of the levee may also be a possible solution. Such a cofferdam must be erected as closely as possible to the water side slope. High cofferdams are not practicable. It may suffice merely to drive a line of piles, in front of which several layers of sand bags are packed.

If it appears that a break in the dike is inevitable, the inhabitants — who should be prepared for such emergency — should be warned as quickly as possible. Nowadays sirens are best for this purpose, but people living in houses at long distances from the danger point must be notified by messengers. Radio is also valuable for this purpose.



Figs. 174 a and b. Collapse of top of embankment

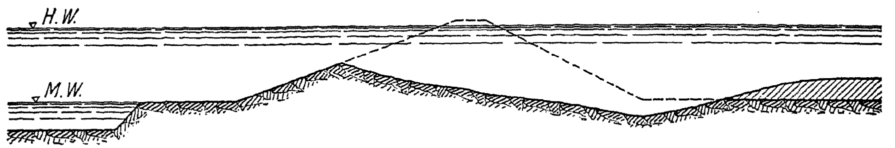


Fig. 175. Usual levee failure

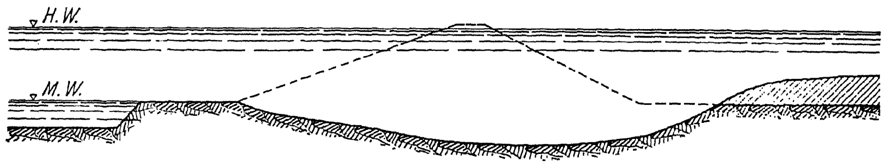


Fig. 176. Foundation rupture

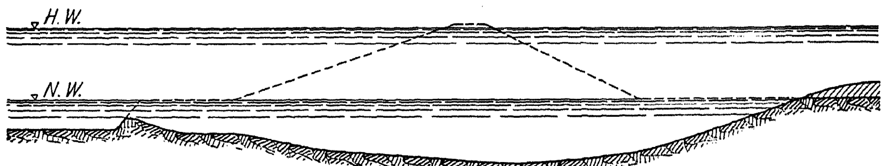


Fig. 177. River bed rupture

Three types of levee breaks may be distinguished from one another:

1. Caving of the crown (Figs. 174 a and b) when only a part of the crown caves in and certain high waters can still be held; also the usual levee break for which mean high-water stages can be held within the channel (Fig. 175).

2. The foundation rupture in which the levee is broken through to the foundation but for which higher foreland still exists so that no water will flow to the lowlands at MW or LLW stages (Fig. 176).

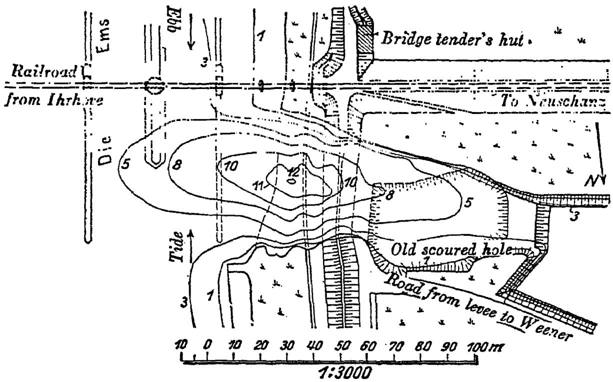


Fig. 178. Effect of storm tide of January 1877. Foundation failure of the Ems River

3. The river rupture (Fig. 177) in which at MW in the river or average tides at the sea, water still flows into the lowlands. Fortunately, this last type of break seldom occurs.

A levee break practically always generates a pool which makes reconstruction at the site very difficult.[1] The new embankment is usually constructed at one side or the other of the pool. In the case of rivers, if the new embankment is built on the water side, it narrows the bed and forms a particularly dangerous stretch. At the sea this is also true because of the projecting corners in the structure. Frequently a dam may not be built at the outer side because the landside slope lies too near to the pool and is in danger of sliding. Usually an inside levee is first built, that is, one landward of the pool. These inside levees, after considerable deposition has taken place in the pool, are replaced by embankments constructed along the line of the old levee. Many river and sea levees have pools behind them which persist as small ponds and are not exposed to the deposition of silt. An inside levee is therefore usually preferable to an outside levee. Fig. 178 shows a levee break on the Ems in which an embankment was first built along the outer edge of the pool, but was later replaced by a permanent levee along the inside.

In the case of river levees, it is not permissible to wait until the water subsides enough so that the land is dry before closing the river

[1] Such pools still exist in East Friesland, Jeverland, etc., as evidences of old storm tides. These may be found far inland; for example, the Grahlman pool at Ostiem (Jever) lies some 8 km. (4.9 mi.) from the nearest coast.

break. (At the sea front the tide occurs twice a day, so that waiting would be of no avail.) A river levee break is generally closed by constructing two wattlework dams and filling between them, or by erecting a temporary quay under the protection of which the new dike may be built.

On the Schelde in 1907 relief was obtained by loading two large sea lighters with rock and sinking them at the location of the break while the tide was changing and the river sinking, thereby forming two rock dams which decreased the current sufficiently so that the levee could be rebuilt.

6. Levee Ramps and Thoroughfares

Levees hinder transportation as well as the normal discharge of inland water. These hindrances can be overcome by thoroughfares and sluices. Levee construction was possible only after the sluice and the levee lock were invented. The possibilities of the sluice must have already been recognized by great races of ancient times. From the fact that open sluices are possible, it follows without further explanation that highway thoroughfares are possible if they are provided with sufficiently safe closing arrangements. Because of the cost of construction, however, passages through the levees have given way to ramps. Furthermore, such passages are usually not as safe as the automatically closed sluices. The general principles of road construction are applicable to the construction of ramps. A levee ramp must be constructed in such a manner that it does not weaken the embankment. Angles which tend to increase the attack from current and waves must be avoided.

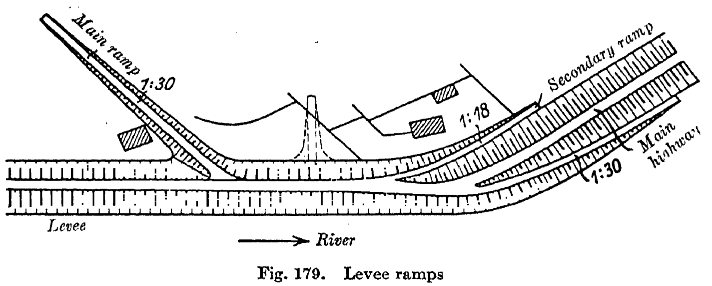
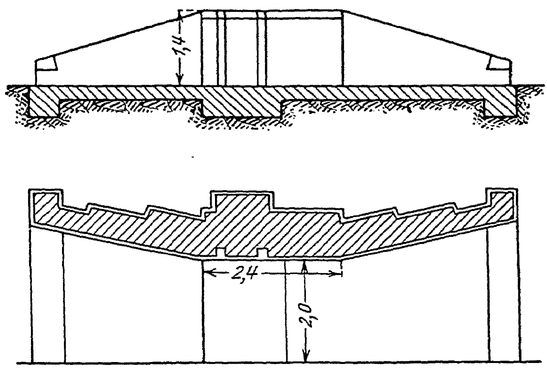


Fig. 179. Levee ramps

The ramp layout shown in Fig. 179 for a river dike has thus been arranged with a ramp leading from top to bottom only in the downstream direction, while on the land side, one directed upstream as well as one directed downstream is provided. The inclination of the ramp is made somewhat dependent upon the height. For important roads, such as those leading to bridges over the river, and on high

levees, the ramp slope should not be steeper than 1:30. For ordinary levees and rural transportation, the ramps may be as steep as 1:10. The choice is somewhat dependent upon the grade of road considered.

Subways through levees are generally constructed with rigid footings and wing walls (Figs. 180 a and b), though it is unnecessary to provide a rigid footing over the entire width of the levee. The best procedure is to drive a line of sheet-piling from wall to wall under the part of the footing which is to receive the gate. In consideration of underscour, the water side of the thoroughfare is thus protected by a rigid floor for the entire distance in front of the line of sheet-piling. During serious high water, the transportation is temporarily stopped. The gap in the levee is usually closed by two beam walls with clay stamped between them. However, for main trafficways it is better to provide the passage with regular lock gates which close automatically even though some water is already flowing through the thoroughfare. Transportation can then be allowed to continue longer and the closure completed very quickly. Danger of dike breaks is greatly diminished by allowing the wing walls to extend about 2 m. (about 7 ft.) into the levee and by laying the floor one-third to one-half the levee height above the foot of the levee.



Figs. 180 a and b. Passageway through a levee

7. Levee Locks; Sluices; and Siphons

a. Determination of Dimensions for Sluices

A levee cuts the water of the inland from its natural course to the river or sea. The sluiceways through levees must be large enough to discharge the quantity of natural flow which is obstructed by the levee; thus, the larger the drainage area, the larger must be the sluice. The water course which leads to the levee is called inner channel; the discharge canal to the river or sea is called outer channel. Every sluice must be provided with gates to protect the lowland against the entrance of high floods. The greater the high floods (storm tides of the sea) the more careful provision for closing must be provided. Many sluiceways

through sea levees are provided with two pairs of gates. One closure suffices for sluices in smaller river levees. The safety locks in localities having dangerous high floods should operate automatically, especially on the sea front. Such locks open automatically at low outer-water stages, and close when the outer water rises above the inner-water stage. Double swing gates with vertical axes have this property when their turning circle is so limited that they cannot lie flat against the wall. In order to back up the inland water temporarily during droughts, that is, for irrigation purposes, an inner gate is frequently added. Open and covered sluices are often differentiated from each other. Concrete, masonry work, iron, and wood are used for construction materials. Timber sluices are further classified into plank, beam, and upright sluices. The locality at which the sluice is to be constructed must be so chosen that as much ground as possible can be drained. Frequently it is impossible to arrange for a single sluice and several must be built at favorable points. The outer channel should be as short as practicable because the steepest slope is attained thereby and the sluice "draws" better.

The type of sluice section and the elevation of the sill are dependent upon the quantity of inland water which must be discharged within any particular period, the stage of water which is permissible in the inner channel, and the controlling water stage in the outer channel. In rivers it is necessary to make computations not only for the lowest water stages of the river, but also for middle-water stages during which drainage is also necessary. The width of the sluice is dependent upon these computations after the elevation of the sluice sill is fixed. Computations for sluices at rivers, or seas not subject to tides, are comparatively simple, but become much more complicated when tidal changes must be considered. In cases where sluices in sea levees also serve navigation, as frequently occurs, they form a part of the ship locks.

Sluices which discharge into seas subject to tidal changes and those which discharge into rivers or tideless seas differ from each other in that tide sluices, with few exceptions (storm tides), are opened twice daily to allow drainage, while other sluices alternately allow continual drainage for long periods and are closed for long periods. They must be supplemented by pump installations if drainage must continue during times of prolonged high floods. The difficulty with tide sluices rests in the short time during which they may be used for drainage. They may be opened for short periods at intervals of about twelve and a half hours; in case the sluice lies over deeply sunken marshes, the period is often very short. Hence the location and dimensions of the sluice should receive special consideration. It is particularly important to have the

outer channel as short as possible for these sluiceways. The longer the channel, the more it is subject to silting. In view of silting, the port sill of the sluice should not be set too deep. It is preferable to make the sluice correspondingly wider. If there is little or no sedimentation, the sill may be set as low as practicable since, for the same area of cross-section, the sluice which extends the deepest has the maximum discharge capacity. The following factors come into consideration in determining the width:

1. The quantity of water to be carried off in conjunction with the discharge capacity of the drainage ditches of the marsh. The quantity of water itself is determined by the quantity of precipitation in the polder and in the higher sandy land belonging thereto.
2. The elevation of the lowest marshlands which drain to the sluice and also the depth of the sill below the inner water stage.
3. The length and depth of the outer channel.
4. The time available for the sluice to discharge the water.

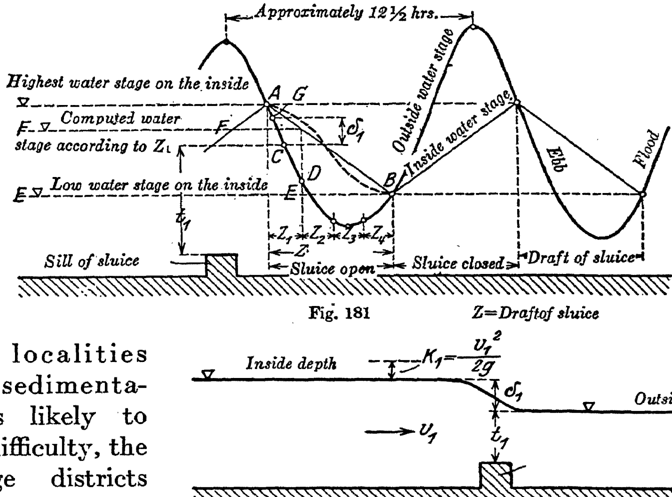


Fig. 181 Z=Draft of sluice

In localities where sedimentation is likely to cause difficulty, the drainage districts should be made as large as practicable for each sluice in order to generate as strong a scouring stream as possible.

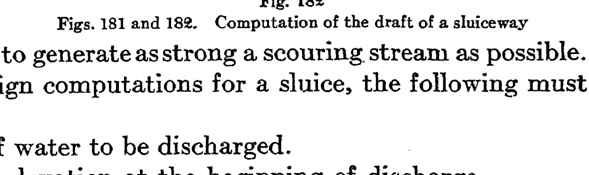


Fig. 182

Figs. 181 and 182. Computation of the draft of a sluiceway

Preliminary to design computations for a sluice, the following must be determined:

1. The quantity of water to be discharged.
2. The backwater elevation at the beginning of discharge.
3. The inside water level at the end of the discharge period.
4. The period of draft of the sluice.

Fig. 181 indicates a study for a sluiceway through a sea levee in a

tidal district. The figure indicates the tide curve in front of the sluice, and superimposed are the inner water stages which intersect the tide curve. When the outer water has fallen to the level A, the sluice is opened. The water then flows outward. The water assembled in front of the levee is discharged, and also the water which drains from the marsh. The water level at the sluice falls more slowly than the sea level, and the latter begins to rise again before the former has reached its lowest point. At the moment that the sea water has risen to the lowest level reached by the inner water, water begins to flow inward. If the sluice does not operate automatically it must be closed at this stage. This moment is represented diagrammatically by point B. The time Z, between A and B, is thus the period of draft through the sluice. It will be observed that it consists of only a portion of the tide period. It increases with the height of the marsh and corresponding permissible height of the inner water level. The period of draft in very high marshes may be as great as nine hours; for marshes of average height, about four and one half hours; and for old, very deeply sunken polders, sometimes only one and one-half hours are available.

During discharge a fall occurs in the sluice. This again is greatest (15 cm. (5.9 in.)) for high ground and for lowland falls to 3 cm. (1.2 in.). Thus the most unfavorable conditions occur in low polders. They operate only for short discharge periods and, having a small fall, they require large sluice openings. Deeply sunken marshes lying far inland usually have the poorest drainage.

The computations for the design of sluices in tidal districts are most readily made by Tolkmitt's[1] method or by a method presented in the *Handbuch der Ingenieurwissenschaften*, Part 3, Volume 7.

Various tide curves come into consideration. The tide curve at spring tide with low low-water does not present the most serious condition to drainage; a curve for mean tides should be used for computations. Cognizance must also be taken of the fact that large quantities of water which flow into an outer channel may considerably vary the form of the tide curve at the sluice. The tide curve in the outer channel in front of the sluice controls the design rather than the tide curve for the sea.

Computation can be made (Figs. 181 and 182) in the following manner: The height of backwater at the sluice can be computed from the quantity of water which flows from the drainage area within the time $Z = 12.5$ hours. In order to provide sufficient drainage of the lands served by the sluice, a definite elevation must be assumed for the low-water level in the inner channel. There is a certain amount of choice

[1] Tolkmitt, *Grundlagen d. Wasserbaukunst*, Berlin, 1898.

in this regard, which is simply dependent upon the discharge capacity of the ditches and upon the form of the tide curve. A choice is possible because the size of the sluice section is to be determined, and from it the quantity of water which can be discharged. The period of sluice draft, Z, is fixed by the position of both the high and low water levels in the inner channel, and the tide curve for the outer channel. This time, Z, is divided into a definite number of small parts, z. In Fig. 182, for example, four divisions have been made. For a total time of the sluice draft of practically eight hours, z was approximately two hours. In this case the division of Z is made in such a manner that the third time period, z_3, lies at both sides of the low-water extremity of the tide curve. Hence the low-water elevation may be considered as the middle outer-water elevation during the time z_3. The total fall is not expressed for the periods z_1 to z_3 by the interval between the inner water level at the time the gate is first opened and the variable outer water level, but by a smaller height since the inner water elevation continually falls. However, it is evident that the greatest fall is available in the period z_1, since the outer water falls to the point D within this period. The fall available during the following periods constantly becomes smaller. The smallest amount of fall develops in the period z_4. In order to determine the quantity of water flowing off during the period z_1 after a first trial computation, it is assumed that the inner water level has fallen to FF in the period z_1 and an approximate computation is made assuming the outer water stage to remain at the mean height C throughout the period z_1 instead of progressively falling from A to D. For the first trial computation, for example, calculations could be made, considering the inner water level unchanged at one fourth of the height AB during the entire period z_1. The same sort of rough approximation is made for the inner water stage. Instead of dealing with a gradually falling inner water stage, the computation is made as though the water had stood at one half height between A and FF during the entire period z_1, namely at point G. Calculations are made with $GC = \delta_1$ taken as the fall throughout the period z_1. Garbe proposes that computations for the discharge per second be made by the formula,

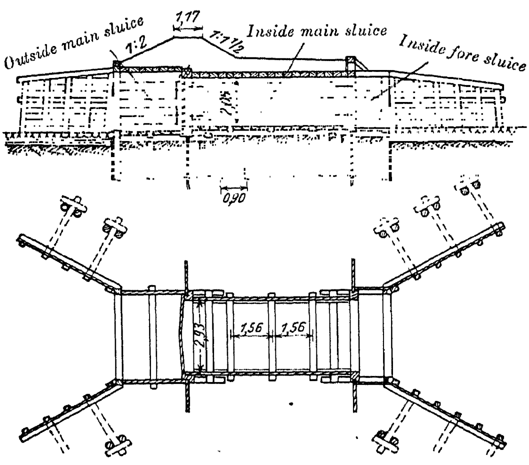
$$q_1 = ub\left(\frac{2}{3}\delta_1 + t_1\right)\sqrt{2g(\delta_1 + k_1)}.$$

In this equation, t_1 is the depth of water over the sill for the assumed mean water stage in the outer channel (Fig. 182) during the time z_1, δ_1 the assumed constant fall during the period z_1, k_1 the velocity head of the approaching water from the inner side toward sluice, that is, $k_1 = v_1^2/2g$ in which v_1 is determined from the discharge Q_1 (which is

to be found) and the cross-sectional area of inner channel. The value b is the breadth of the sluice at its narrowest section, and u is a coefficient of discharge which may be taken as .85 to .95 for very smooth sluices, but may have a value below .60 for unfavorable circumstances. Thus the first approximation of the discharge for an assumed breadth b, is $Q_1 = z_1 q_1$. The result will correspond to the assumed discharge only by chance; b must be varied until there is agreement with the assumed value of Q_1. Calculations are then made for the remaining partial time periods z_1 to z_n, due regard being given to the fact that any change made in b also results in a change in the previously computed discharges Q_1, Q_2, etc. The latter accounts for the reason for dividing Z into but few subdivisions.

β. Construction of Sluices and Sluice Siphons

Small sluices up to about 1.5 m. (4.9 ft.) width and height are called pump sluiceways: Practically all pipe sluices belong to this group. Old sluiceways of this nature are all of wood construction. Figs. 183 a and b illustrate a simple beam sluiceway constructed of wood. Wood may be used only when there is assurance that the structure will not rot out, hence when it is continually kept sufficiently moist by the earth surrounding it. Wood pipes are used considerably nowadays. They are especially good in that they follow the settlement of the levee very well because of the elasticity of wood. In order to hinder settlement, the usual types of sluice structures are often set upon pile underpinning. For small sluiceways a row of piles along the axis is sufficient. Small sluices are also made of iron pipe, concrete pipe, and stone. They must have a self-closing trap on the water side so that sufficient closure will take place automatically in case of backwater from the outside. In many instances it is advisable to provide additional safety by installing a sluice gate. For this portion of the installation, the reader is referred to the discussion in "Weir and Lock Construction."



Figs. 183 a and b. Timber sluiceway

Figs. 184 a and b show the temporary wooden sluiceway for the Marien sluice in Oldenburg, this structure being used until the construction of a masonry sluice could be completed. Fig. 186 shows the masonry sluiceway in section and an end view. The entire structure is very compact. It has about five times the area of the wooden sluice.

The Hoyer sluice at Husum (Figs. 187 a to d) is another example of a masonry sluice. This is a type of construction which is seldom used, being a combination of an open levee-lock with covered sluices at both sides. The lock, which has a clear width of 7.5 m. (24.6 ft.), serves

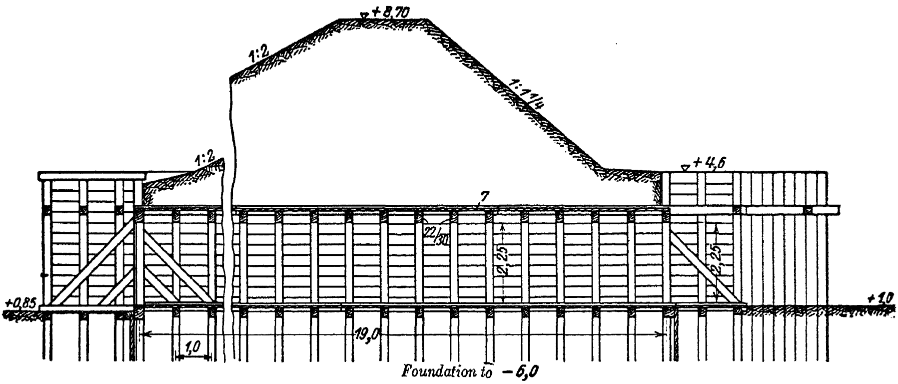


Fig. 184a. Longitudinal section

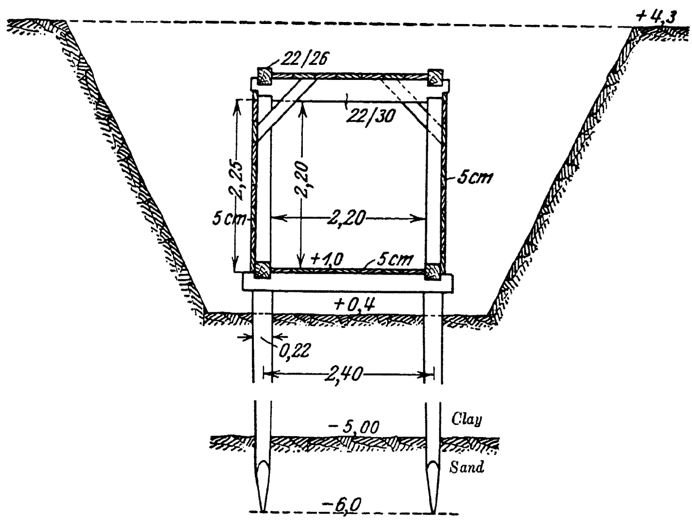


Fig. 184b. Cross-section of sluice and construction ditch

Figs. 184a and b. Layout of the Marien timber sluice

navigation purposes as well as drainage. The example was chosen because it illustrates nearly all factors which must be observed in connection with such structures. The entire structure rests on piles. The large lock possesses two gates on the outside and one gate toward the inside. All of these are mitering gates. The covered sluices are closed off against the outer water with mitering gates which rest in grooves at the top and bottom. The mitering develops the water seal only for the middle joint. The inclination of the port sill is, therefore, very flat (Fig. 187).

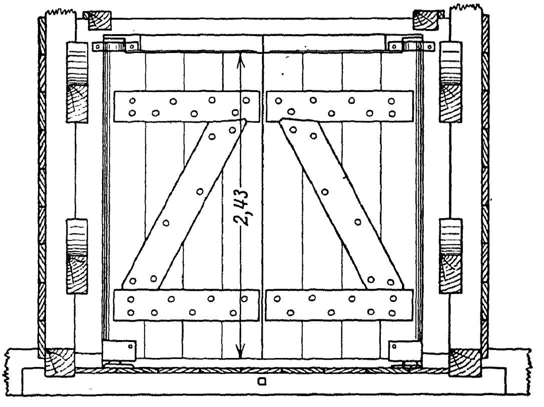


Fig. 185a. Elevation of gate of sluiceway

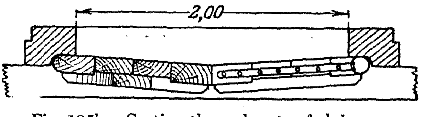


Fig. 185b. Section through gate of sluiceway

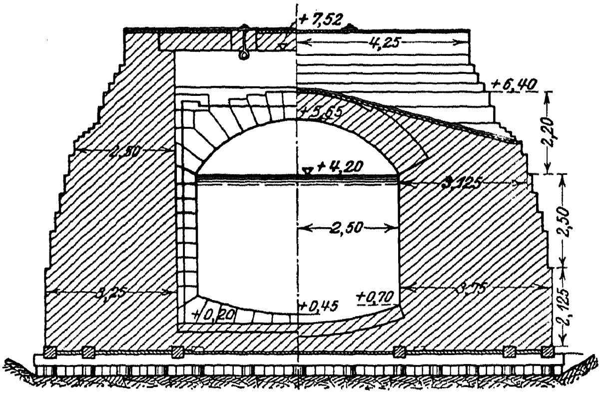


Fig. 186. Cross-section of the new Marien sluice. The left half shows the outer elevation; the right half a cross-section through the center of the sluice

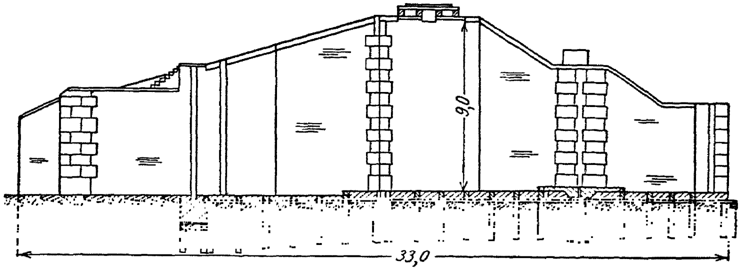


Fig. 187a. Longitudinal section through the open sluice

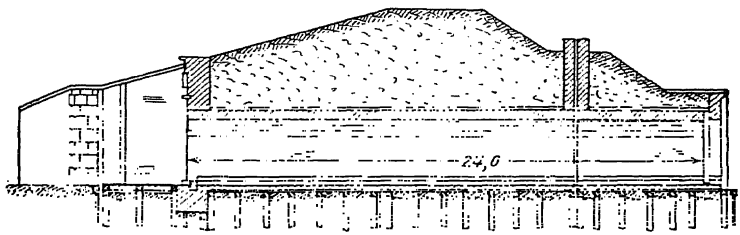


Fig. 187b. Longitudinal section through the covered sluice

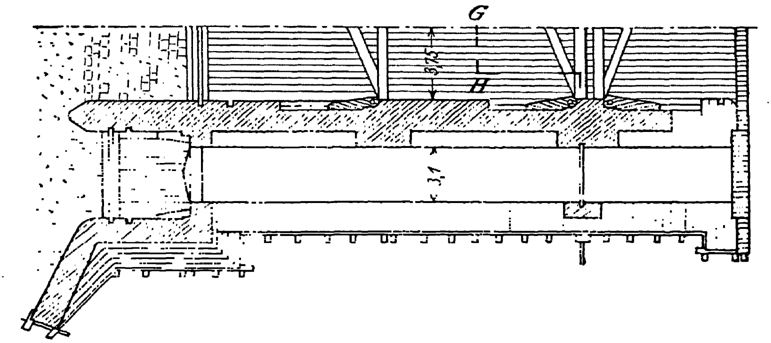


Fig. 187c. Half plan

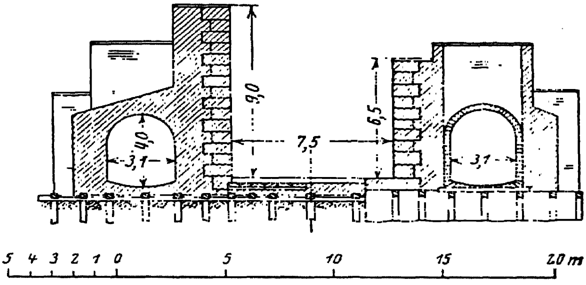


Fig. 187d. Cross-section

Figs. 187 a to d. The Hoyer locks at Husum

The inner gates of covered sluices are replaced by sluice gates which are operated by windlasses from the inner berme of the structure. Fig. 188 shows a type of trap door used for the closure of a cast iron pipe sluice. The entrance and exit to the sluice must invariably be framed in masonry work, including wing walls and a curtain wall, so as to preclude underscour.

Piercing levees with sluices endangers the embankment at the location of the sluiceway. Water can readily flow along the side of the sluice and cause springs to develop. Abraham therefore arranged siphons to replace the ordinary type of sluice. These siphons follow the side slopes of the dike so that they can not be the cause of a break in the levee. The operation of siphon sluices is dependent upon the removal of air. For this reason a jet pump was arranged to keep the siphon free continually from air. Figs. 189 a to d show such a siphon sluice.

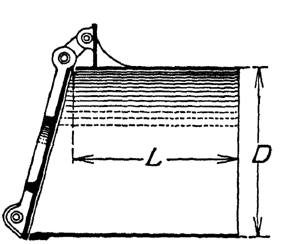
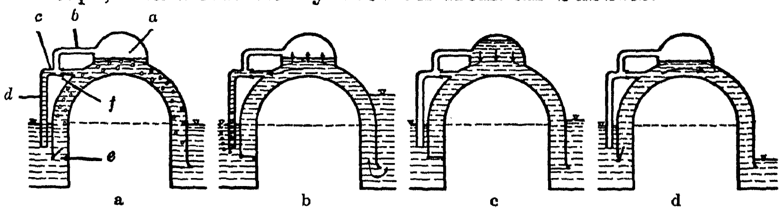


Fig. 188. Pipe sluice with trap door arranged with double hinge for $D = 20$ cm., 30 cm., 40 cm. $\}$ $L = 40$ cm.

In the figure, *a* denotes the storage space for water; *b*, a suction tube; *c*, a pipe (of only a few centimeters diameter) which sets the jet pump, *f*, into operation; *d*, a drain pipe of the pump for the water mixed with air; *e*, a trap which is closed by an excess pressure from the outer water. The combined action of the jet pump, *f*, and the dome, *a*, keeps the siphon from going out of commission during the long pauses between two drain periods of the sluice. The manner of operation is as follows:

(*a*) The siphon is filled with water and the outer and inner water elevations are approximately the same. The trap, *e*, is opened, the siphon stops, and *a* fills slowly with air from air bubbles.



Figs. 189 a to d. Abraham siphon sluice

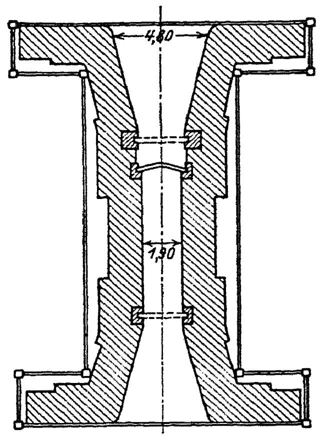
a Water stage inside and outside. b Rise of the outer water, actuation of the air pump and drawing out of the air in the dome. c Siphon ready for operation and sinking of the outer water. d The siphon begins to operate the instant at which the water stage on the outside is deeper than on the inside.

(*b*) The outer water rises further and by its excess pressure closes the trap, *e*, and sets the jet pump, *f*, through the pipe, *c*, into movement. This causes air to be drawn from the dome, *a*, through *b*, toward *c*, and causes *a* to become filled with water.

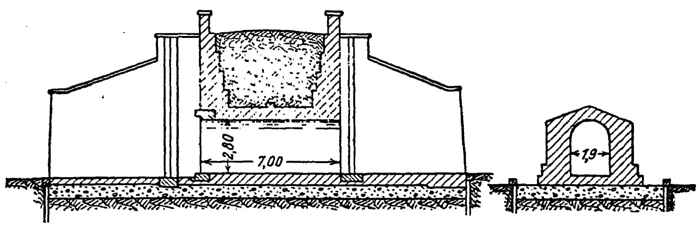
(c) The outer water level falls again and the excess pressure has become too small to keep the jet pump, *f*, in operation. The dome, *a*, becomes filled with air and gives off water. The dome, *a*, must be arranged in such a manner that it can hold all of the air rising during the time of approximately equal inner and outer water pressure.

(d) The outer water falls further and as a result of excess pressure of the inner water, the trap, *e*, is opened and the siphon goes into operation.

Large sluices are constructed as open or closed and long or short structures. Open sluices are necessary when the structure becomes so high that the weight of the levee above it is not great enough to over-



a



b c

Figs. 190 a to c. Short sluiceway at Hassum

a Horizontal section. b Longitudinal section
c Cross-section through the center of the sluiceway before being covered with earth fill

come the buoyancy. Short sluices (Figs. 190 a to c) must be closed off by vertical forewalls. Long sluices are preferable to short ones; large sluices are always set on pile foundations if good supporting ground does not lie high enough so that the masonry body can be set directly on it. Heavy masonry sluices should preferably be constructed of concrete and stone. Reinforced concrete sluices can be made at practically the same cost as wood sluices, hence there is no reason for using wood sluices which readily rot and must be renewed. *Wood sluices, because of rotting of the wood, have often been the cause of levee breaks. They are always to be considered as weak points in levees and are to be very carefully watched.*

Sluice closures present nothing particularly new compared to closures

for ship locks. Reference is simply made to the fact that for covered sluices, such as at Husum, the mitering gates should have a rebate above and below so that breaking through the gates need not be hindered by a comparatively sharp mitering inclination of 1:3 or 1:2. These gates are given only a weak mitering ratio of 1:6 to 1:10, so that, because of the heavy mitering pressure, an excellent water seal is attained, and destruction of the wood due to breaking through is not possible.

PART FOUR — WEIRS

A. GENERAL

a. Definitions[1]

In the following discussion the term *weir* is used in its broader sense. Weirs form a part of the class of structures which, in general, are grouped together under the name barrages. Under barrages may be included valley dams, weirs, river dams, groins in rivers, dikes of every type, etc. Valley dams are intended to assemble and store water and equalize the discharge. Weirs, on the other hand, have their principal purpose in raising the water level in rivers; that is, concentrating the river fall at individual points either to increase the water depth upstream or to raise the water level to such an extent that sufficient operating fall for hydroelectric plants is obtained. Hence weirs are differentiated according to whether their purpose is to create a greater river depth or to raise the water surface. For the first purpose, a preliminary stipulation is that sufficient space exists to be filled with water; for the second, the water conducting section may be very small, the only requirement being to raise the water level. In both classes of weir layouts, one is not concerned with assembling large quantities of water but with the creation of depth or fall.

Neither dams nor weirs can serve their particular purposes without also acting in part as the other type of structure; that is, dams raise the water level while weirs invariably also collect water, though usually in small amounts.

The purely legal definitions of the Prussian water law are insufficient for practice. With regard to valley dams the law states: "For barrages in which the height from the bed of the water course to the crown of the structure is more than 5 m. (more than 16 ft.), and the collecting basin to the crown of the damming structure has a capacity of more than 100,000 cu. m. (more than 3,540,000 cu. ft.) (valley dams or *talsperren*), the following stipulations are applicable." From this it follows that all barrages which have a height from the river bed to the crown of more than 5 m. (more than 16 ft.) and those which, when filled to the crown, have a capacity of more than 100,000 cu. m. (more than 3,540,000 cu. ft.) of water, are called dams. According to this definition, practically all

[1] German terminology is somewhat different from the American with reference to this subject. In general it is more descriptive and makes finer distinctions. The translator has, as far as possible, retained the descriptive form of the nomenclature so that the individuality of the original work may not be lost. Amplifications have been made where necessary.

weirs constructed in the Weser, Elbe, Rhine, and other large German rivers were dams. Dams, however, which were constructed in a side portion of the mountains and did not obstruct a water course were, according to the law, strictly taken, not dams. Hence, legal terminology has little in common with the purpose of the structure.

b. Purpose and Classification of Weirs

Raising the water level over weirs may have for its purpose one or more of several different results. Usually several results are to be attained for a single development. Among these may be mentioned:

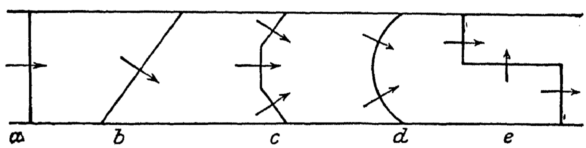
1. Power development; 2. Navigation; 3. Irrigation and drainage; 4. Equalization of discharge (flood control).

The first purpose, power generation, is served by practically all of the older weirs in small rivers. Such barrages were already erected in the early Middle Ages when they were constructed in the form of fixed weirs. At present many of these are being replaced by movable weirs, the latter being arranged to operate automatically so as to retain the advantage of the fixed weir. The second purpose, navigation, is served by most weirs in larger rivers, many of which have also been constructed for power development. Nowadays most weirs constructed in the larger rivers are primarily for power development and the improvement of navigation plays a secondary rôle.[1]

For the fulfillment of the third purpose, in Germany, weirs are usually constructed only in small rivers. In large rivers the interests of agriculture often enter as a secondary consideration.

The fourth of the above divisions is to be considered only as a secondary purpose. This effect is naturally obtained to a certain degree in newer weirs by the installation of movable weir openings, but may be particularly required in the interest of agriculture and industry.

The effect of weirs upon the discharge of rivers is discussed in the



a Straight weir b Oblique weir c Broken weir d curved weir e Offset weir

Fig. 191. Plan view of various types of weirs

section on canalization of rivers. The profile of the water surface arising upstream of a weir is a concave backwater curve.

Structurally, weirs may be divided into two large groups: fixed weirs and movable weirs.

[1] However, there are many exceptions to this rule in America.

Weirs are also distinguished from each other according to the manner of overflow, and may be classified as chuting weirs and overfall weirs. Chuting weirs are designed in such a manner that the overflowing stream adheres to the body of the weir; in overfall weirs the water nappe pulls away from the weir body. Chuting weirs have a flat apron toward the tail-water; overfall weirs have a steep drop on the tail-water side. (Figs. 200 and 201.) These dynamic differences occur among both fixed and movable weirs. The various types are illustrated in Fig. 191 and the effect of the weir form upon the discharge is discussed in section e.

c. The Weir According to the Prussian Water Law

Mention has already been made of the definition given for dams by the Prussian Water Law, and correspondingly the definition given for weirs. At present, authorization of the State authorities is required for all new weirs and all changes in existing weirs. In Prussia the authoritative body is called the District Committee (*Bezirksausschuss*); in case of adverse decisions, it is permissible to bring the issue before the Court of Appeals in Berlin.

Objections to allowing water power development or the construction of a weir may be divided into those of the resident above and those of the resident below the installation. According to Ludin, the principal objections are:

a. On the part of the upstream resident:

1. When the proposed structure causes backwater up to a mill lying further upstream and thereby causes a decrease in head for the mill.

2. When floods occur.

3. When the rise in the ground-water level would cause swamping of lands, flooding of cellars, and the like.

4. When high-water discharge is hindered and damages are incurred on drainage ditches, canalization, etc.

5. When deposition occurs in the river bed, thereby causing increased danger to levees at times of high-water.

6. When the natural water regulation of a river is replaced by artificial regulation of a weir, thereby making the tail-water of a power plant further upstream dependent upon the arbitrary service at the new weir.

7. When the amount of water at disposal is decreased by evaporation or seepage.

b. Objections from the inhabitants below the installation may be due to:

1. Sinking of the river bed because of further wandering of the

detritus banks, the timely replacement of which is hindered by the weir, resulting in deepening of the bed and sinking of the tail-water level and the adjacent ground water.

2. Danger to pile foundations and to bridge piers which have not been grounded deep enough to be safe when the tail-water level sinks.

3. By-passing of water through the headrace and tailrace channels, thus laying dry a long stretch of the tail-water bed, depriving the settlers adjacent to the river of stock-watering facilities, and damaging fishing possibilities and the beauty of the district.

c. Objections from residents both upstream and downstream may be in common when the movable weir temporarily causes an arbitrarily large variation in flow.

d. Discharge at Weirs; Effects above and below Weir; Buoyancy

A weir should be designed to discharge water in such a manner that the velocity is diminished sufficiently within the space of the structure so that there is no danger of serious scour in the tail-water channel.

In overfall weirs, it is attempted to attain this condition by allowing the water to fall vertically into a natural tumble-bay, so that most of the excess velocity head is dissipated by impact. This type of construction requires particularly resistant tumble-bays. Some attempt has been made to overcome the difficulties of this method by developing the chuting weir. Here the water flows as a shallow stream of high velocity (Fig. 208), but usually decreases its velocity surprisingly fast. The energy of the stream is consumed by the formation of a "hydraulic roller"; the latter develops for overflow weirs as well as for chuting weirs. (See the hydraulic roller diagrams in Figs. 205 to 208.) These rollers have the same effect as friction bodies. They consume energy in rotating and diminish the velocity of the discharging stream. At the point of contact, the direction of the roller is the same as that of the main stream. A hydraulic roller which lies above the main stream, accordingly, maintains a direction of flow at its upper surface which is opposite to the direction of the stream. Rollers lying below the stream turn in the reverse direction. In designing a weir, consideration should be given to attaining energy-consuming rollers.

A weir dams up a stream, thus causing the formation of the backwater curve. Beginning with a point far upstream at which the backwater effect is no longer noticeable, the wetted section continually becomes larger and the velocity decreases as the weir is approached. The scouring force likewise decreases so that at some distance upstream of the weir the transporting force of the water is too small to move the detritus. Banks of coarse detritus gather here and the bed of the stream

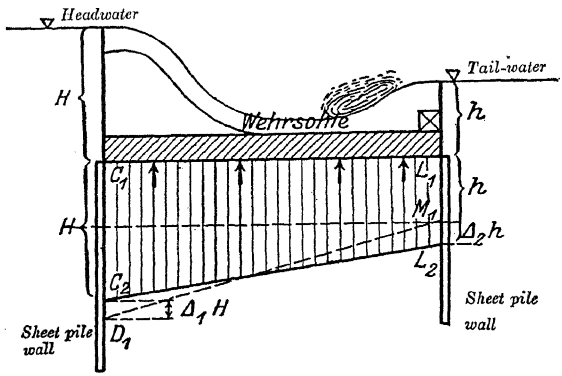
is raised higher and higher, until finally conditions upstream become similar to the former conditions. The detritus movement is then again in equilibrium. In some instances, detritus has even been transported over the crest of the weir, but in any case, care must be taken in selecting resting places for the detritus. It frequently happens that, when a location has been chosen for assembling and dredging detritus, the detritus comes to rest much further upstream than was expected, and disturbs navigation.

Downstream from the weir, until equilibrium takes place, the reverse condition readily develops. The river bed does not vary before construction of the weir. The discharge, slope, and cross-section are at first the same as formerly. The movement of detritus continues just as before except that there is no addition of material to replace that carried off, so the river bed is deepened with consequent damage. The damage may become very serious, because, although equilibrium again develops upstream, still more time is required before the downstream part of the river again reaches a state of equilibrium. Sediment will deposit and after a time the diminished slope will again become steeper. A new state of equilibrium develops only after the slope again becomes great enough so that the detritus which wanders to the tail-water end is transported further. It is not certain, however, that this condition will ever occur.

The ground-water level upstream is frequently raised, particularly when the backwater at the weir rises above the ground-water elevation. In the case of sandy ground, artificial waterproofing is practically impossible, but in the course of time a natural waterproofing layer develops. At first large quantities of water trickle through the bed and side embankments, swamping adjacent territory. Some relief may be obtained by drainage. If the river carries fine clayey silt, this material will force its way into the pores and gradually clog them. In the case of the Weser weir at Hemelingen near Bremen, within a few years after its construction, it was observed, from ground-water gages, that the ground-water level in the river bed a short distance upstream from the weir was only slightly higher than the tail-water stage. In the Spree upstream of the Muhlen dam above Berlin, the ground-water level lies some 2 m. (some 7 ft.) lower than the surface of the Spree. The time until this condition is attained gives occasion enough for many disputes; in most cases, to be sure, sealing occurs of its own accord, but the time required cannot be accurately estimated.

The buoyancy under the weir body is largely dependent upon the elevation of the ground-water level above and below the weir. The most serious conditions occur during the early stages after construction. In permeable ground a line of sheet piling should be placed at the upstream

end of the weir. Frequently the same is done at the downstream end although a double row of piles would be better at that end. A line of sheet piling causes a reduction in the amount of hydrostatic uplift. Hence (Fig. 192), the uplift under a weir floor is the same regardless of whether the weir is fixed or movable. At the downstream side of the upper line of sheet piling, the buoyancy force will be an amount $\Delta_1 H$ smaller than the pressure H of the headwater, measured from the surface of the headwater to the lower edge of the structure. It will be an amount $\Delta_2 h$ greater than the water pressure below the lower wall. However, it is very difficult to predict the magnitude of $\Delta_1 H$ and $\Delta_2 h$. By building gage wells into the structure, the water stages at both sides of the line of sheet piling can be measured after the weir has been erected. Dr. Enzweiler[1] measured a difference in pressure of over 2 m. (6.56 ft.). Differences of such a magnitude are most probably unusually high for the amount of sinking in ground-water level at lines of sheet piling. They are the result of a comparatively rapid ground-water movement. The hydrostatic pressures under a weir depend entirely upon the permeability of the substrata and particularly upon the height of the backwater. If a head of 6 to 8 m. (19.7 to 26.3 ft.) is encountered upstream of a slab of say about 12 to 15 m. (39.4 to 49.2 ft.) in width, then a difference to probably 1 m. (3.28 ft.) may be expected between the sides of a line of sheet piling. The effect is usually unfavorable for the slab, since the lower portion of the slab is not loaded as heavily as the upper. The more uniformly the uplift is distributed, the stronger the downstream portion of the plate must be. Hence, it is advantageous to avoid placing a line of sheet piling at the downstream end, a permeable row of piles being preferable. The piles will merely hinder underscour. If the self-sealing action, and therefore impermeability of the upstream bed increases considerably, the uplift under the slab continually becomes smaller until eventually the buoyancy practically disappears and the



Wehrsohle = Weir floor

Fig. 192. Hydrostatic uplift under the floor of a weir.

[1] Enzweiler, *Die Grundwasserabsenkungsmethode in ihrer Anwendung auf den Unterwassertunnelbau unter besonderer Berücksichtigung der Gross-Berliner Verhältnisse.*

hydrostatic pressure approaches that of the tail-water head. In general, it may be assumed that the static conditions of a weir constantly improve. If it maintains equilibrium as a new structure, the subsequent conditions will be much better, provided that the concrete does not deteriorate or that other similar difficulties do not occur.

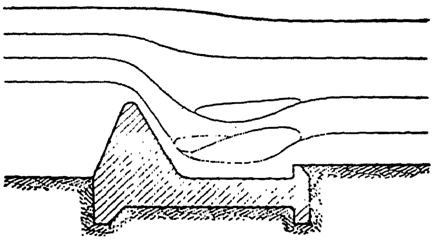


Fig. 193. Effect of a weir upon the water stage

The effect of a weir upon water stages, regardless of whether it is fixed or movable, becomes smaller the higher the river stage. Fig. 193 illustrates the circumstances which arise. A weir which develops a head of 4 m. (13.1 ft.) at LW, may cause only a small swelling in the river at HHW. During the latter stage a very small fall suffices to cause enough increase in velocity to equalize the cross-section caused by the weir.

e. Location of Weirs

The cost of a foundation generally increases with the depth to which it must be sunk. The deeper the cross-section to be closed by a weir, the smaller it is, since the mean velocity in a deep cross-section is larger than in a flat one. If there is choice between a narrow, deep section and a flat, wide one, then from many points of view the wide, flat section is to be given preference. The area closed will usually be larger, but foundation costs will be less. Furthermore, a wide cross-section is available for high-water discharge, so that the influence of the weir will be less noticeable than would be the case for a narrow section. Since damages to the upstream resident may easily cause lawsuits, the seriousness and cost of which cannot be estimated, it is usually remunerative to keep the backwater as small as possible at high-water stages. The backwater can be diminished for a large number of water stages by lengthening the crest of the weir, but, not for HHW. Some advantage is here obtainable by constructing diagonal or polysided weirs. Diagonal weirs may be of safe design if precaution is taken to maintain good connection with the shore at the side of the weir extending furthest upstream. Generally the water will be discharged in a direction normal to the weir. Consequently one shore may become endangered (Fig. 191). If the shore next to the side lying farthest upstream is protected by rip-rap, etc., this sort of layout is satisfactory. Many diagonal weirs are in use. Curved weirs should preferably be curved upstream in the middle, because of the increased safety, and, since the water is

thereby conducted to the center of the stream, shore protection is unnecessary. Sufficient protection should be provided for the river bed below the spillway. Polysided weirs, to be sure, have a longer crest than curved weirs, though they are seldom used because of the high cost. In case of very high-water stages, the length of crest of the weir no longer plays a noticeable rôle. For these water stages the river width at the weir is the deciding factor rather than the length of the crest of the weir.

B. FIXED WEIRS

a. General

Hydrostatically, fixed weirs may be classified as follows:

1. Overflow weirs (Fig. 195), in which the weir crest is always above the tail-water level.

2. Submerged weirs, in which the crest lies below the tail-water. Submerged weirs develop from overflow weirs when the discharge increases to the extent that the tail-water rises above the weir crest (Fig. 196).

3. Submerged sills in which the top edge of the sill is always lower than the tail-water level (Fig. 197). These play an important rôle in river regulation.

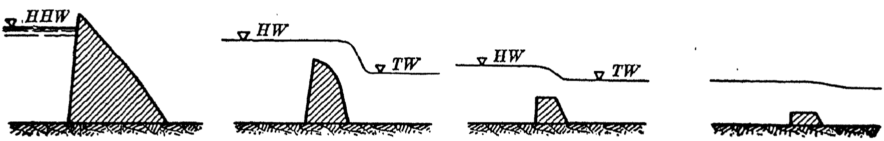


Fig. 194. Dam Fig. 195. Overflow weir Fig. 196. Submerged weir Fig. 197. Submerged sill

HHW=High headwater HW=Headwater TW=Tailwater

Figs. 194–197. Forms of damming devices

Dams (Fig. 194) which are never to be overtopped, of course, need not differ from weirs in the form of construction, but are treated separately under the heading of dams.

As to the nature of construction, weirs may be classified in the following groups:

1. Those of fill material, usually called permeable weirs.

2. Those of mass masonry or concrete, usually called impermeable weirs.

3. The multiple type, including various kinds of hollow dams.

The division according to permeability or impermeability usually has little meaning, because many weirs of mass masonry construction have, in the course of time, become permeable and weirs of rock-fill

may become so well compact that they are practically impermeable. Weirs constructed of wooden sheet piling (Fig. 221) with clay filling have all the characteristics of a cofferdam and do not fit into the group called permeable weirs.

Barrages of brush work, sticks, or rock-filled coffers do not, in the strict sense, classify as weirs but as stream obstructions. They are not constructed to raise the water level but to stop the movement of heavy detritus and to transform the bed of the valley into a waterfall. These construction works have been discussed in the section on Obstruction of Torrents.

b. Rock-Fill Weirs

Weirs made of rock-fill may be very pervious for simple structures but, by applying good paving over the surface and a sheet piling core-wall, they may be made practically impervious. The simple form of

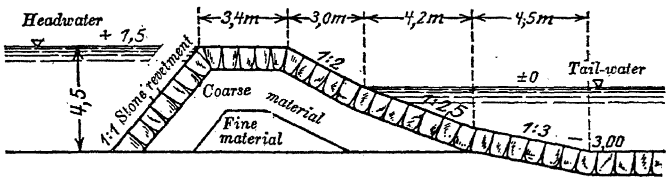


Fig. 198. Rock-fill weir with stone revetment

rock-fill weir was developed by nature inasmuch as occasionally this type of rock wall is formed in rivers, causing backwater at low- and middle-water stages. In constructing such weirs, rocks are either hand-picked, the smaller ones being placed in the middle and the coarser ones at the outside of the structure, or the rock is used without segregation into sizes. In the latter case more rock is required but the weir becomes heavier and more resistant. Although possible, it is very difficult to give a fill weir a good form for the discharge of water. Almost always the structure will develop into a chuting weir in which there is dangerous turbulence at the tail-water bed. However, by the use of rows of piles or intermediate lines of sheet piling, it is possible to improve the form by resolving the downstream side into steps. (Fig. 199 shows such an improved form.) In this case, four rows of round piles have been provided, and the crest is paved in order to prevent the tearing away of individual stones

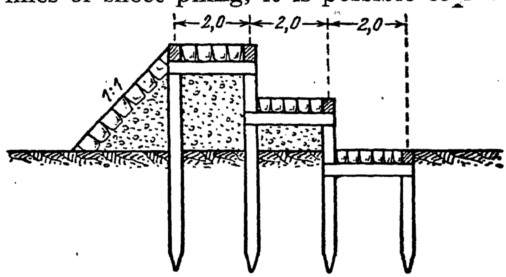


Fig. 199. Fixed weir with stone revetment

from the weir. If a line of sheet piling were placed so as to reach the highest point of the structure, the weir would be impervious.

c. Mass-Masonry Weirs

Various forms of mass-masonry weirs are shown in Figs. 203, 204, and 218 to 220. These are usually very simple forms of weirs, many of them having existed in rivers for centuries. Usually they are grounded between lines of sheet piling or rows of ordinary piles. The weirs are constructed of concrete or masonry. Because of underscour, many of them have settled and become pervious in the course of time. In some instances at LW stages, distinct eddies occur in the tail-water, which are due to the flow of water through the cracks caused by unequal settlement.

Many investigations have been made with regard to the most favorable form of gravity weirs. Notable experiments were performed on the fixed weir in the Moldau on Hetz island at Prague, and by Rehbock on models at Karlsruhe, Germany. Only three of the weir forms which were investigated in the Prague experiments are given here (Figs. 200 to 202). Several intermediate designs are not included. The experimental work was begun by studies of a chuting weir, and then was broadened to include various forms of overflow weirs, of which the last form is here described. In this form the water adheres to the back of the weir. The models were constructed to a scale of 1: 20, the dimensions for the full size structure being given in the figure.

In the first experiment, a large pool formed downstream of the apron. An experiment was then made with a longer apron (Fig. 220). This experiment indicated that lengthening the tumble-bay resulted in merely moving the pool farther downstream, although the decrease in size amounted to less than ten per cent.

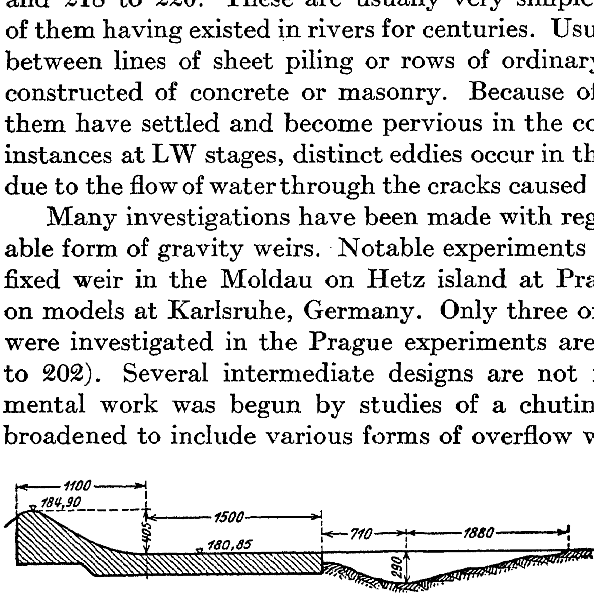


Fig. 200. Chuting weir having a long apron

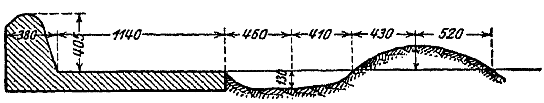


Fig. 201. Overflow weir without a buffer wall

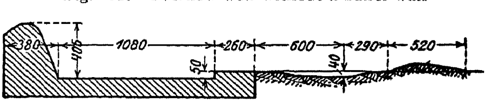


Fig. 202. Overflow weir with buffer wall

Figs. 200–202. Experiments on the Moldau weir at Prague

Subsequent forms indicated a significant increase in the size of the pool, both in depth and length. The chuting form of the weir was then discarded and an overfall type chosen (Fig. 201). The scour was greatly reduced thereby.

The next form of overfall weir (not indicated here), having vertical downstream wall and free nappe, was not as good as the first of this

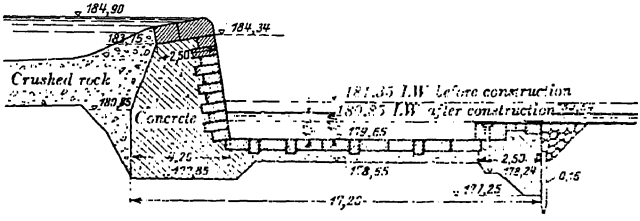


Fig. 203. Helmer weir in the Moldau River at Prague

type; the pool became longer and deeper. This was probably due to the fact that the friction on the back of the weir was lacking in the second case, the water falling through air. The form shown in Fig. 201 was again tested but with the end of the tail slab improved by the addition of a sharp-edged nose at the end of the tumble-bay (Fig. 202). Experiments with the nose raised higher resulted in further deepening of the scour. The type of construction used is that shown in Fig. 203.

Fig. 204 shows a cascade form of weir which was constructed in the Elz at "Wasser." Practically the same effect could probably have been

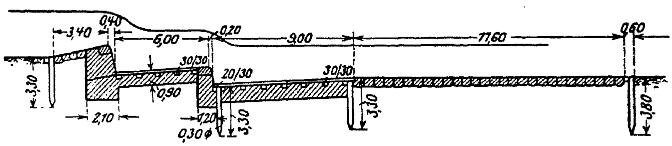


Fig. 204. Elz weir at Wasser

obtained by constructing a buffer sill instead of the cascade. As a result of various experiments, especially those by Rehbock, considerable change in the design of overfall weirs has been adopted. Rehbock's experiments indicated that a correctly designed chuting weir with a well-placed buffer sill will develop hydraulic rollers having sufficient capacity to dissipate the energy of the chuting water. In the first experiment (Fig. 205) the bottom roller under the stream is large, the surface roller small; the bottom roller scoured a deep pool in the bed. Enlarging the sill (Fig. 206) enlarges the surface roller. The best design for the chuting weir (Fig. 208) resulted in the formation of two surface rollers, one of which was very large. More recent experiments by Rehbock indicate that the water is quieted still further by adding a dentated strip at the lower end of the apron. It appears certain that any further improvement will be the result of making the surface roller as large as possible. Probably the best means for accomplishing this

end (Fig. 209) consists in providing a mild rise in the apron from the bottom of the spillway to the tail-water bed and adding a dentated sill at the end of the apron.

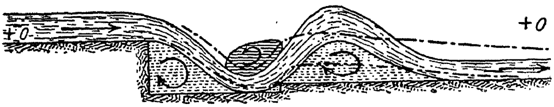


Fig. 205. Short tumble-bay provided with a buffer wall. The roller vanishes periodically (— • — line)

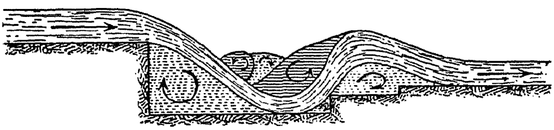


Fig. 206. Tumble-bay with two buffer walls

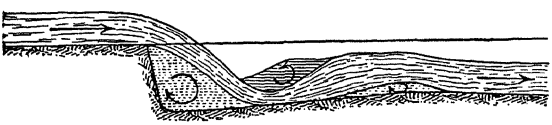
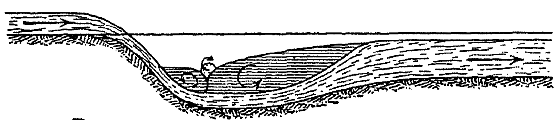


Fig. 207. Tumble-bay without buffer wall



5 m. height

Fig. 208. Tumble-bay with counter floor

0 10 20 *30 meters length*

Figs. 205–208. Roller for chuting and overflow weirs conducting equal quantities of water (experiments by Rehbock)

Another good method of quieting the water, especially for raft chutes, consists of the use of movable grills such as beam grills (so-called raft beds). Figs. 210 to 217 show the results of Swiss experiments, including velocity diagrams for weirs with and without raft beds. The first two diagrams indicate the effect of the raft bed in causing the desired velocity distribution. The other diagrams show the scour effects. In this group of studies the design equipped with a raft bed suffers the least scour. This layout (Fig. 217) was erected at Basel.[1]

An older form of chute which is used for very high heads amounting to 20 m. (65.6 ft.) or more is shown in Figs. 218 and 219. This is the design used for the weir in Drac River at Avignonette. The form is very similar to that for dams, though the layout probably would have been improved by adding a buffer sill at the end of the apron. Experimentation on models of structures of this nature, however, was developed long after the weir had been constructed. In this type of design it is necessary to carry the apron a long distance downstream from the weir crest, although the apron was sloped upward in order to displace the point of scour as far downstream as practicable.

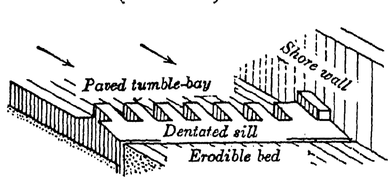


Fig. 209. Dentated sill by Rebbock

[1] *Schweiz. Bauzg.*, 1918, p. 25.

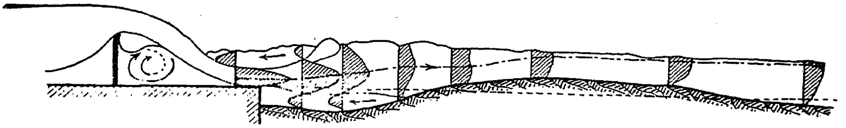


Fig. 210. Velocity distribution in the scoured pool

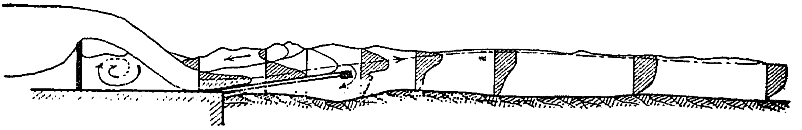
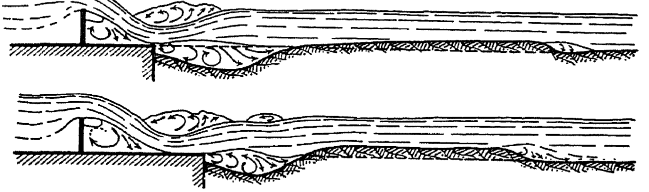


Fig. 211. Velocity distribution when a raft is hinged to the end of the tumble-bay



Figs. 212 a and b. Pool formation, tumble-bay without buffer wall

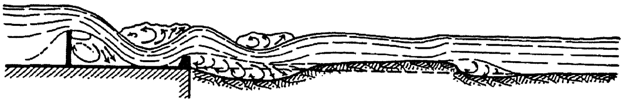


Fig. 213. Pool formation, tumble-bay with buffer wall

Fig. 214. The same with longitudinal ribs

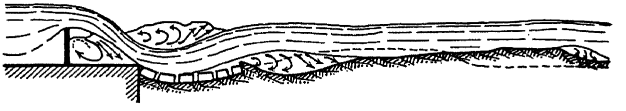


Fig. 215. Revetted bed

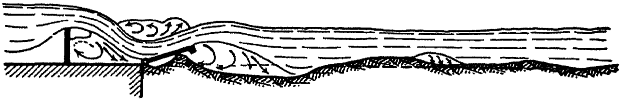


Fig. 216. Flat plate for spring board

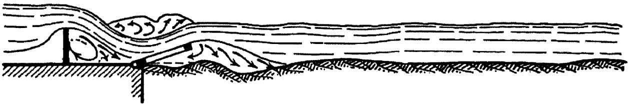


Fig. 217. Grid system as raft spring-board

The structure is considered a weir because its primary function is not that of collecting water; structurally, it is classified as a dam.

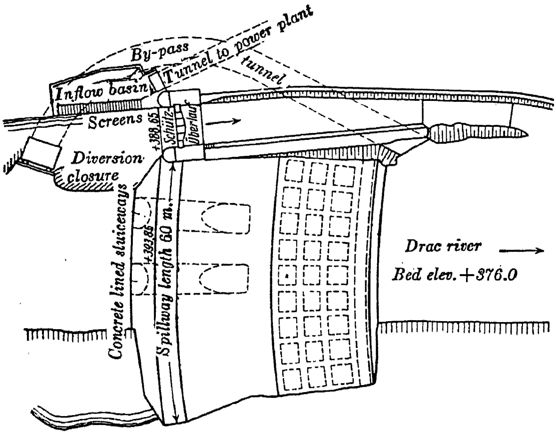


Fig. 218. Weir in the Drac River at Avigonnet

Although of primitive form, the weir at Rheinfelden (Fig. 220) is to be considered a chuting weir. It is built upon a rock foundation and therefore did not demand special precautions with reference to scour. However, even rock substrata may lead to great difficulties during erection. The weir was constructed by beginning from both shores and working toward the middle. Thus the flow was so highly concentrated and strengthened at the open pass that, before the closing section was placed, the bed was eroded to the extent shown in Fig. 220. The original bed was at the elevation of the dotted line. The difficulties which arose because of the pool and the means of surmounting these difficulties are discussed by Rehbock in the volume on weir construction of *Handbuch der Ingenieurwissenschaften.*

d. Weirs Constructed of Sheet Piling; Hollow Structures with and without Fill

Weirs constructed of modern sheet piling are completely impervious. When built of the older form of wood or of reinforced concrete piles, they are impervious only when backfilled by a dense material.

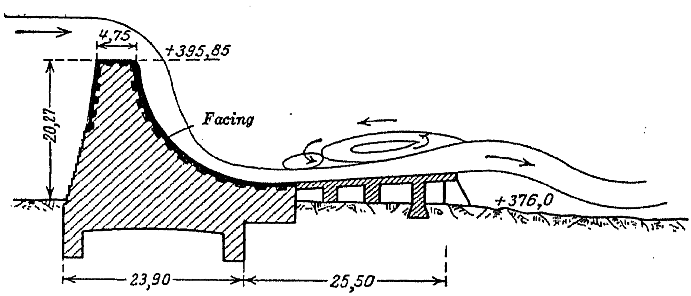


Fig. 219. Cross-section of the Drac weir

Up to the present time, very few pure sheet pile weirs have been constructed, but weirs of sheet piling with clay fill have frequently been constructed. A section of such a weir is indicated in Fig. 221. Usually three rows of sheet piling are driven; one at the upper side, one at the

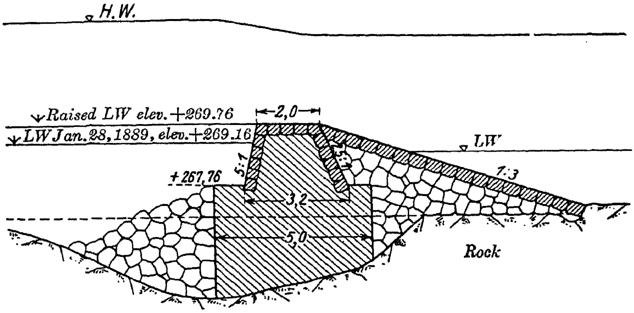


Fig. 220. Weir at Rheinfelden at point of scour

lower side, and one under the highest point. These weirs are carefully constructed, filled with clay, and topped off with an impervious cover. The structure is similar to a very carefully constructed cofferdam. Special care must be given toward obtaining a compact cover, because otherwise the clay will become eroded. If well constructed, these weirs are absolutely water-tight.

Hollow weirs of reinforced concrete: Weirs of this type have been constructed in large numbers in America. Sheet-pile foundations, which have been frequently used for these weirs, have caused difficulties. The underscour thus occurring showed that multiple reinforced concrete structures are very tough.

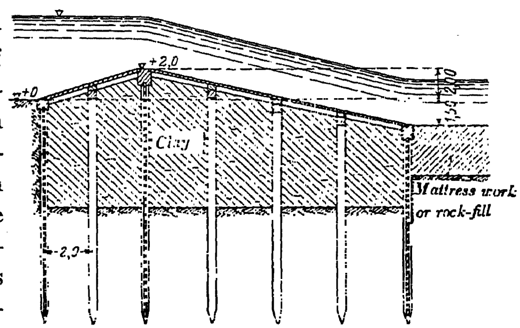


Fig. 221. Fixed weir made of rammed clay and covered with wood

Hollow weirs differ fundamentally from other fixed weirs in that the upstream wall must be inclined if the structure is to be economical. The slope of the upstream face makes possible a downward hydrostatic pressure which replaces the lack of dead load in the body of the weir. The water pressure acting upon the sloped slabs may be resolved into horizontal and vertical components. Thus, since the normal pressure[1]

[1] The forces W_w, W_s, and W_n, intersect at the upper surface of the slab only when the slab is a plane.

at any point does not vary, the vertical component may be varied by varying the inclination of the slab. For a flat slab, the resultant horizontal and vertical components of the water pressure, W_2 and W_1,

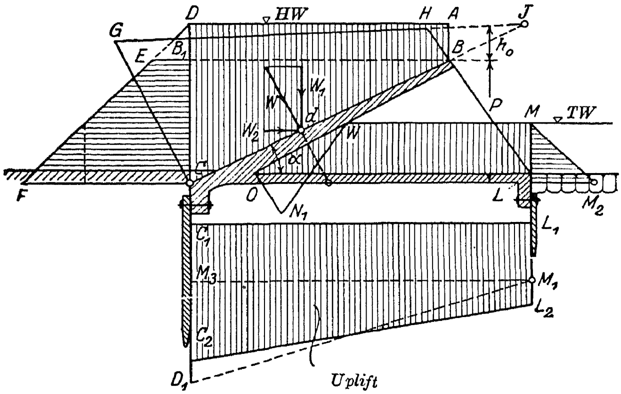


Fig. 222. Computation for an open hollow weir

respectively, intersect in the upper surface of the slab. Each of these forces and its resultant is represented diagrammatically by the trapezoidal areas CDAB, FEBC, and CGHB, respectively, and W_1, W_2, and W. These forces must be combined with the dead load of the weir. The force, W, increases as the height of water increases, while the dead load of the weir, G, remains unchanged. From this it follows that the resultant of all forces, R, is nearest vertical at the lowest water stage. As the water rises, the force, W, is displaced forward since the center of gravity of the inclined trapezoid lies further upward. Consequently, it is possible to make the distribution of pressure on the footing slab more uniform. If the slab is made long enough so that at low water the center of pressure lies in the vicinity of the back kern point, as the water rises, the force W will be displaced further toward the middle and the edge pressure increase less than when W intersects the base at the middle in the beginning. The effect of tail-water is usually neglected for ordinary water stages. It has a buoyant effect on the weir. The tail-water rises more rapidly than the headwater and consequently the buoyant effect increases more rapidly than the pressure from above. The tail-water effect must be taken into consideration for high-water stages. For safety against sliding, the angle β at which the resultant, R, intersects the vertical, is of importance. The value β is dependent upon the value of α. Experience indicates that the front slab should make an angle of $\alpha = 38°$ to $45°$ with the horizontal. The corresponding angle β is then greater than $52°$ to $45°$.

Because of the buoyancy due to tail-water counterpressure, the most unfavorable condition for the weir will be at a middle high-water stage. This stage must be particularly determined. In no case may the resultant, R, intersect the base outside the kern. This requirement is usually easy to fulfill.

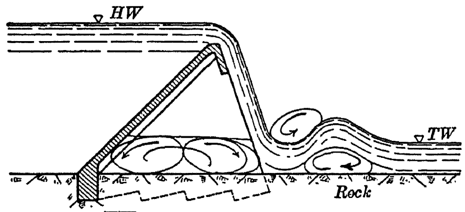


Fig. 223

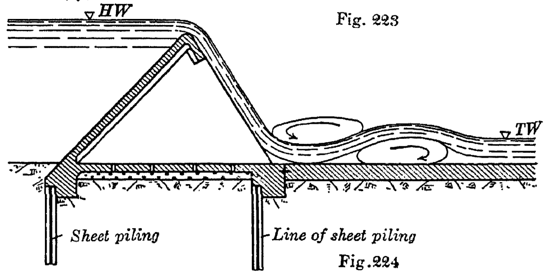


Fig. 224

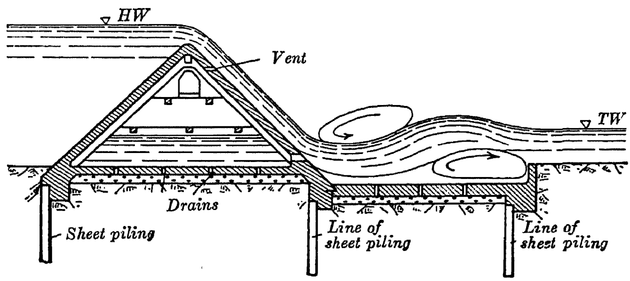


Fig. 225

Figs. 223–225. Hollow weir of reinforced concrete

Hollow weirs are usually constructed as open or closed weirs. Figs. 223 and 224 show forms of open weirs; Fig. 225, a form of closed weir. A foundation slab is usually not necessary in case the structure is built on rock strata. The pier need simply be widened sufficiently to avoid exceeding the carrying capacity of the river substrata. At locations where the substratum is soft, a continuous foundation slab is constructed, though each pier may be set on a separate slab footing and these footings may be supported on piles. The comparative costs will be the deciding factor. Foundations without continuous footing slabs are

to be preferred because such footings will not be subject to an appreciable amount of hydrostatic uplift. The slabs on the upstream face of the structure are usually constructed as simple beams on two supports. The thickness of the slab increases from top to bottom, and this is usually true of the pier (Fig. 226). In the case of small weirs the interval between piers is usually between 1.5 and 2 m. (4.9 and 6.6 ft.), but for high-head structures the interval is made as much as 6 m. (19.7 ft.) and more. The thickness of both the slab and the supporting weir varies between .2 and 1 m. (between .7 and 3.28 ft.). The most economical spacing of the piers may be determined after definite assumptions have been made concerning the carrying capacity of the ground. The concrete mix ordinarily used in the piers averages about 1: 3: 6; that in the slab, about 1:2 :4. The slab must have a rich mix in order to make it impervious.

Where gravel or crushed rock can be obtained cheaply, it may prove economical to fill the structure with this material. In the latter case the thickness of the piers may be diminished because of the increased safety against buckling provided by the rock fill. Too great faith should not be laid in the drainage of water from under a continuous slab foundation because the drains readily become clogged. It is advantageous, however, to provide drains for a new weir, because there is little question regarding their effectiveness in the early life of the structure. Drains are usually less necessary as the structure becomes older, because in the course of time the bed on the headwater side usually becomes more impervious of its own accord. Hollow weirs have been designed

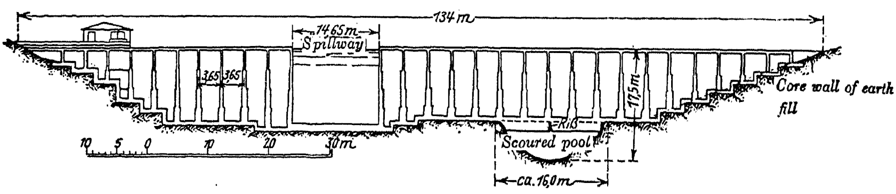


Fig. 226. Hollow weirs at Pittsfield (U. S. A.)

to dimensions approaching those of small dams. Some of the designs provide for streets, water conduits, and, in one instance, even a ship canal going through the body of the weir. The structure is well qualified for such arrangements because it is hollow.

An interesting foundation failure of a weir occurred at Pittsfield (Fig. 226). The weir was completely undermined along a 16 m. (53 ft.) length but did not break. A settlement of 30 cm. (12 in.) took place and was again raised by jacks. A wall was first constructed of solid rock.

C. MOVABLE WEIRS AND SIPHONS

a. Classification

Poirée invented the movable needle weir in the year 1834. Up to that time the fixed weir provided the only means of damming up large rivers. Small movable weirs in the form of mill passes and raft passes were known already in ancient times. The surmounting of water steps by means of locks was also possible after the invention of the chamber lock. The frequent serious disturbances in the water cycle of a river, even by fixed weirs, always had to be taken for granted until the invention of movable weirs. The disturbances were particularly dangerous at times when ice came down the river, because ice blockades often developed as a result of fixed weirs, and frequently caused breaks in the river levee and the submergence of extensive lowland areas. The devastation of harvests in consequence of rapidly rising summer high-water was always a danger. All of these damages have been overcome largely by the development of movable weirs.

Movable weirs must meet the following requirements:

1. Greatest possible watertightness should be provided if the water is to be used for power development or irrigation. If the water is not to be used at the weir, the structure need only be tight enough to provide the desired rise in the water elevation.
2. The mechanism must be simple and easily operated.
3. It must be possible to open the weir completely at any time.
4. In rivers carrying ice it must be possible to open the top part of the weir so that ice can be conducted off without lowering the headwater level appreciably. Simply raising the body of the weir does not suffice, because floating ice will not submerge to flow through the bottom of the gate opening.
5. The gate should be arranged so that the bottom may be raised part way, allowing sediment to be scoured away. Furthermore, there should be no difficulty in closing the gate again.
6. All mechanical or moving parts should lie in such position that they can be investigated and painted periodically. It must be possible to lay the weir dry temporarily for this purpose, or the parts must be located so that they become dry of their own accord at low-water; otherwise, it must be possible to raise the movable parts out of the water.[1]

[1] Rollers and the like often become immovable, not only under water but often also after several months motionless position above water. Such a condition was of late (1925) experienced at Dorverden on the Weser. Another even more dangerous occurrence is the grinding action of sand, especially where the water squirts through narrow intermediate spaces.

Movable weirs fall into the following groups:

a. Lock and canal emergency closures.

b. Bar weirs.
   1. Stop-log weirs.
   2. Needle weirs.
   3. Curtain weirs.

c. Gate weirs.
   1. Trap weirs.
      α. Weir traps which are opened by water pressure but which must be closed again by mechanical devices.
      β. Weirs which are opened by water pressure but which close again by counter weights.
         The Doell trap.
         Crest weirs.
      γ. Weir traps which operate entirely hydraulically.
         Ordinary two-armed traps.
         Drum or rotary weirs and angle-sluice weirs.
         Double-trap weirs.
   2. Sluice weirs.
      α. Older sluices for mill channels.
      β. Newer weirs with movable intermediate supports.
      γ. Newer weirs without movable intermediate supports.

d. Cylindrical weirs.
   1. Segment weirs which are operated mechanically.
   2. Sector weirs which are hydraulically moved.
   3. Roller or complete cylinder weirs.

e. Siphon spillways and the like.

As is indicated by the above divisions, movable weirs occur in a great variety of forms, but most of them do not play an important part in modern developments. The majority of those in the first groups are antiquated types. The double-trap weir, the sluice weir without intermediate support, and particularly the cylinder weir, are continually developing into the weir forms of the future. This trend is explained by the simplicity of their construction and operation, and their consequent low cost of installation and maintenance. The group, *a*, called lock and canal emergency closures, are in a class by themselves. Nevertheless, they do not require particular discussion but can be described along with the individual weir forms. The particular individuality of this group lies in the fact that they are often out of use for months and even for years and then come into operation for only a short time.

b. Bar Weirs

1. Stop-Log Weirs

Wooden stop-logs are rectangular hewn beams which slide up and down grooves or rabbets in columns for checking the flow of water. Several forms of stop-logs are shown in Figs. 227 to 230. The arrangement for joining them together — whether as hooks, eyelets, or bolts — is of little importance. Stop-logs are frequently used to dam off channels. The lowest stop-log should rest on a flat bed; a sill is not to be recommended because it catches dirt or detritus. The laying of stop-logs in flowing water, which backs up only after the beams have been placed, is a simple operation. More difficulty is experienced in bringing the beams into deep water because buoyancy must be overcome. In the latter case the stop-log must be forced down at both ends by vertical bars which are pushed downward by winches. Another method consists in fastening a block and tackle at the bottom so that the beam can be pulled down (Fig. 231). Very few of this type have been built. A

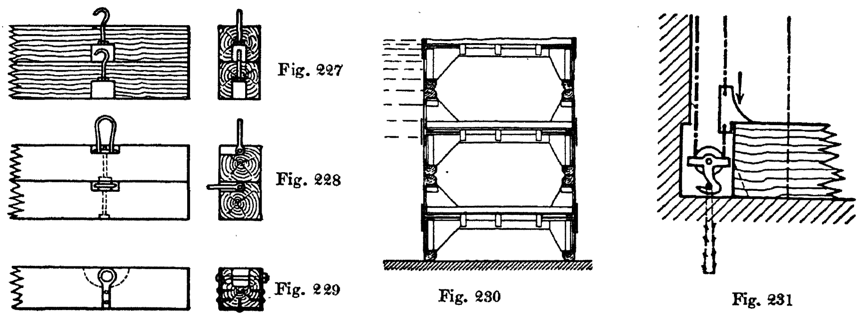


Fig. 227 Fig. 228 Fig. 229 Fig. 230 Fig. 231

Figs. 227–231. Stop-log weirs

Fig. 227. With hooks. Fig. 228. With rings. Fig. 229. With bolts for raising and disconnecting the beams. Fig. 230. Steel stop-log designed by M.A.N. Fig. 231. Appliance for dismounting stop-log.

large one was built in the Ottawa[1] river in Canada, but can not be considered especially advantageous. The timber resources of the country probably had some effect upon the choice in this case.

Stop-logs have been used principally for emergency closures for locks but have been largely replaced by needle-weir closures. Stop-log walls have also been constructed with collapsible rebate columns. One method is to hold the rebate column fast by a chain at the top and let it be movable at the bottom. Upon loosening the chain, the column folds over, allowing the stop-logs to float freely. It is best to hold them to the fixed column by a chain (Fig. 232). A recent development of

[1] Rehbock. *Handbuch der Ingenieurwissenschaften.*

stop-logs in the form of roll stop-logs has been devised by a German steel fabrication company. The idea involved was first published by Theodore Hoech.[1]

For large locks, the length of the stop-logs causes difficulty. Beams are either built up as steel box girders having a waterproofing wall on one side, or hollow, reinforced concrete beams are used. In general, stop-log weirs may be considered as an antiquated type.

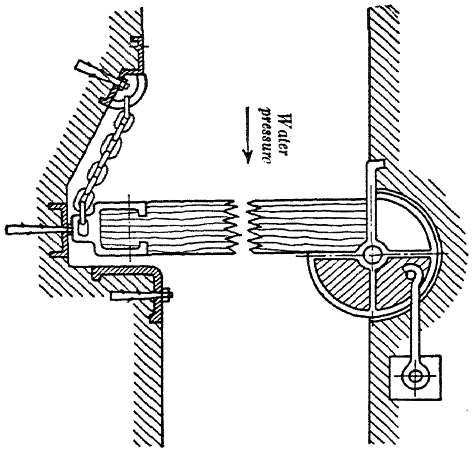


Fig. 232. Stop-log weir provided with a revolving column for rapid opening

2. Needle Weirs

a. Development of Needles

A needle weir consists of individual beams held erect and leaning against a support at the weir bottom and against the lower edge of a steel service bridge at the top.

A section of a needle weir with fixed trusses is indicated in Fig. 233. These trusses must be spaced at such an interval that the sum of the truss costs, together with the needle rests and service bridge, is at a minimum. The nature of the construction of the trusses and the form of the bed slab of the weir requires no particular discussion. The needle rest may be a steel beam or a reinforced concrete beam.

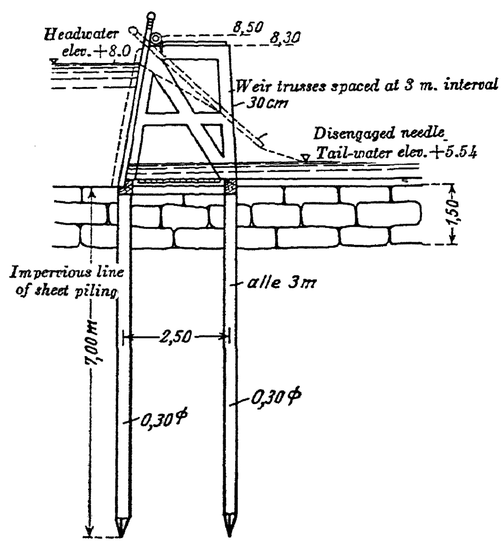


Fig. 233. Needle-weir truss on piers of reinforced concrete

The needles are of importance in the development of the weir. They may be of wood or of Mannesmann steel pipes. The needles should be so formed that

[1] Detailed information regarding the roll stop-logs is published in the trade pamphlets of the *Maschinenfabrik Augsburg-Nürnberg.*

they can readily be placed by two men. Lightness of the needles, therefore, is as important a requirement as strength. Hence, with wooden needles it is permissible to stress the material to the breaking limit, the diminution of strength of wood under water being taken into account. In section, the needle is a planed rectangular plank which, in case of long needles, is thickest at the point of greatest bending moment. The top end is developed into a handle so that the needle can be placed from the service bridge by one man. The needles rest in front of the bottom rebate, and are placed as close to each other as possible. Each needle is pushed against the adjacent one with a ramming rod. The intermediate space remaining between the needles amounts to about 1 cm. (.4 in.) in 10 cm. (4 in.) width. These intermediate spaces are caulked by throwing coal ashes or other similar material over the surface of the weir. After a short time this material, which acts as a filter, becomes so impervious that hardly any water penetrates. Pipe needles were used successfully at Doerverden, where a rope about an inch thick was used for caulking. This rope was fixed at the lower end of a long iron bar, by means of which the rope was pulled down to the lower end of the needle.

The stress in the needles is computed by considering them as beams resting on two supports. The maximum moment, because of triangular loading, is below the middle. It is generally assumed that the water surface extends to the needle rest, and tail-water is neglected. If the angle which the needle makes with the vertical is α, then it will usually be satisfactory to assume $\cos^2\alpha = 0.9$. The somewhat cumbersome formula for the thickness, d, of the needle at the point of greatest bending moment, then reduces to the simple form

$$d = \sqrt{\frac{0.043\ h^3}{\sigma}}$$

where d is the thickness of the needle in the direction of the river in m., h, the height of the needle rest above the lower support in m., σ, the allowable stress in the wet wood in kilograms per sq. cm. In order to obtain light-weight needles, the value σ is usually chosen at the highest limit, so that every year a large number of broken needles must be replaced. A value of $\sigma = 200$ kilograms per sq. cm. (2845 lbs. per sq. in.) for picked timbers is sometimes exceeded. This was done, for example, in Lucerne. There the height of the needle above the bottom support is 2.8 m. (9.2 ft.) and the needles are 6 cm. (2.4 in.) thick; that is, the fiber stress allowed is equal to $\sigma = \frac{0.043}{d^2} h^3 = \frac{0.043}{0.06^2} 2.8^3 = 265$ kilograms per sq. cm. (3775 lb. per sq. in.). Hence, for especially good wood, the

value $\sigma = 200$ kilograms per sq. cm. (2845 lb. per sq. in.) may be used. Wooden needles begin to become inconvenient at heads of 3.5 to 4 m. (12 to 13 ft.). The use of Mannesmann steel pipes, first installed at Hemelingen near Bremen, is a significant step forward because such needles may be made much longer, a head of 5.5 m. (18 ft.) being attainable. For this head, the needles have a diameter of 21.5 cm. (8.5 in.) and a wall thickness of 6 mm. (.23 in.). They are closed at the top and bottom, provided with a hole to allow entrance of water at the bottom, and have an air cock at the top. They are floated into place and then allowed to fill with enough water so that they float vertically and can be handled in the same manner as wooden needles. The watertightness is at least as good as for wooden needles. The smaller area of contact is more than compensated for by the uniform shape of the needles. Rope caulking is very effective.

In order to make it easier to take out wooden needles, they are provided with hooks. A hooked needle is shown in Fig. 234. The hook is at a projection, making possible the introduction of a lever from the service bridge. The hooks are long enough so that they still remain on the needle rest when the bottom of the needle has been raised above the lower support. They are pushed out by the current but remain hanging on the round needle rest above. In order to make fine regulation of the weir possible, Greve invented the so-called Greve hanger (Fig. 235) in connection with the Fulda canalization. Every other needle is supplied with such a hanger. By means of the lever, *d*, it can be pushed forward so that it lies against the flapped over hanger; thereby transforming the previously impervious weir sheet into a grating without making it necessary to remove the needles. If the high-water wave subsides, the needles can again be brought into their old position.

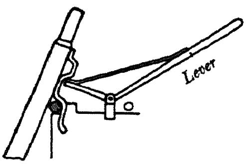


Fig. 234. Guillemain hooked needle ejector

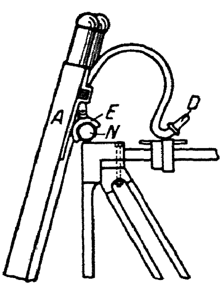


Fig. 235. Needle ejector by Greve

The invention of Kummer, a Belgian engineer, in which every individual needle rest reaches only from one truss to the other and which can be flapped loose in a horizontal plane downstream so that the needles lose their upper support and fall over, has not proven entirely successful. The sudden loosening of a large opening between trestles causes an impact in the weir endangering the entire structure. The Kummer method is not treated here as it is considered out of date. Removal of

the needles is made easier at the Fulda installation by the use of an overhead rope crane spanning from pier to pier and supplied with a simple hook for raising the needles and laying them on a small transport car.

β. Movable Needle-Weir Trusses

Movable needle trusses have undergone great development since the time of Poirée. Fig. 236 shows a front view of a needle weir with movable trusses of the old form in which individual trusses lay on each other. Fig. 237 shows a new form in which each truss is independent, thereby making it possible to open any arbitrary weir section.

The movable needle truss consists of a hinged frame which is movable at the bottom. When the trusses are erect, a trap spans from truss to truss and serves as a bridge. The practice is to start work at the uninhabited shore, laying over one truss after the other. In order to make possible laying over the first truss, the land pier must have a groove into which the first truss may rest. This is indicated in Fig. 236. The trap thereby lies on the truss. The chain on which the trap is held while being laid over is then fastened to the next truss, and thus, after taking away the needles, the entire series of trusses may be laid over. In raising the trusses, inasmuch as the beginning of the chain to each succeeding truss is raised above water, little difficulty is encountered in raising the entire series of trusses. More accurate sketches of this type of construction are unnecessary, since weirs with such narrow trusses are no longer built because they are uneconomical; the construction and service cost is too great and the time required for laying over too long.

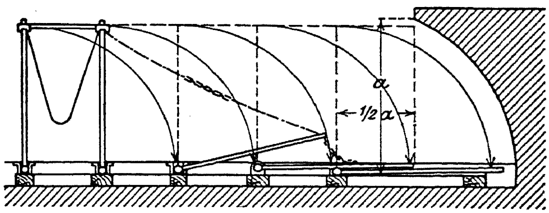


Fig. 236. Needle weir with the old form of truss arrangement

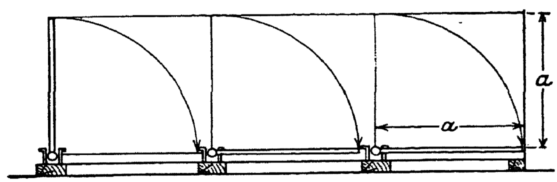


Fig. 237. Needle weir with the new form of truss arrangement

Fig. 238 is a sketch of a newer type of needle weir used in the canalization of the Moldau at Wegstädtl. Trusses of the old type originally welded together by Poirée are used here except that they are made up of standard sections which are riveted together. There is no lower transverse tie, the lower portion being developed as a portal so that the trusses can lie inside each other when laid over. No truss can be

arbitrarily laid over because its head will strike the adjoining truss which still stands. The trusses lie flat on the bed and allow less area for the detritus to attack, but they are still seriously endangered by wear. The distance between trusses is 3 m. (10 ft.) and the truss height from 3.3 to 4.8 m. (10.8 to 15.7 ft.). Neither the height of head nor distance between trusses has been changed to the extent that the weir presents any innovation. The bridges were not constructed as traps; the individual bridge frames, which carry the pipe-shaped needle rest, are taken off and brought to the shore before the trusses are laid over. In operation, however, these weirs are very much better than the old arrangement. With the old Poirée weir having 1.25 m. (4.10 ft.) interval between trusses, three men required two hours for the laying over of one truss; hence, sixteen hours, or forty-eight man-hours, for a 10 m. (33 ft.) weir opening. In Wegstädtl ten men are needed for one hour to open a weir of 10 m. (33 ft.) length. The added speed of operation is particularly valuable.

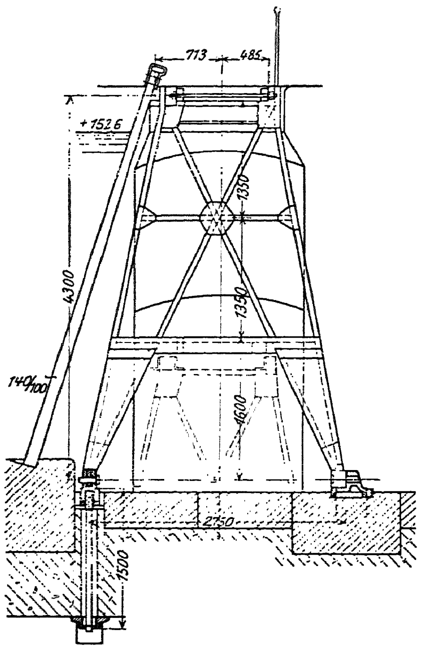


Fig. 238. Needle-weir truss at Wegstaedtl on the Elbe. The lowered truss is shown with a broken line.

Trusses are usually laid over with the help of fixed windlasses. A really great advance in needle dams has been made in the United States by increasing the interval between trusses so that every truss can be laid over independent of others in the series. Truss intervals of over 6 m. (20 ft.) with heads between 5 and 6 m. (between 16 and 20 ft.) have been built. It is possible by the use of pipe needles to develop still higher heads. In the United States the heavy needles are drawn by windlasses. Increasing the interval between trusses has resulted in reducing the cost of the weir about 25 per cent.

In Germany at present needle weirs are considered unsuitable for navigable rivers of the north. There is constant danger of the needles freezing together at times of sharp frost. It is very difficult to loosen them under such circumstances. The attendant, because of worry of freezing weather, will either lay over the weirs too soon — then if no

freezing occurs he did wrongly — or, in order to hold the head as long as possible, he will let the weir stand. If the works then freeze together, there is danger of flooding if high water happens to develop at the same time. Hence, needle weirs should be used *only in rivers without ice* dangers as they are inferior to most newer weir types.

The A-frame weir,[1] an outgrowth of the needle weir, is an interesting and unique arrangement, though somewhat complicated. It employs a collapsible weir frame which is also a needle or damming shield.

3. Curtain Weirs

The attempt to make the needle weir water-tight by rolling canvas over the upstream face stimulated the invention of the curtain weir. This weir (Fig. 239) consists of a curtain of wood staves which are tied to each other by bronze links. The curtain, which is constructed quite similar to a modern window curtain, rolls on a heavy cast cylinder suspended on the bottom stave of the curtain. The roller is wound up by a double rope. The curtain can either be supported on weir trusses similar to those for needle weirs or on standards which are hung from above on a bridge and rest against a bearing at the bottom. Several weirs of this nature have been constructed. This type of weir is very expensive and out of date, and does not merit further description.

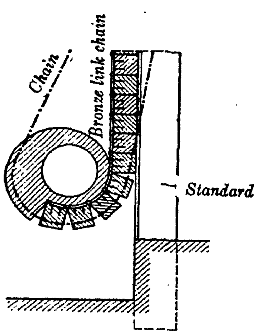


Fig. 239. Curtain weir

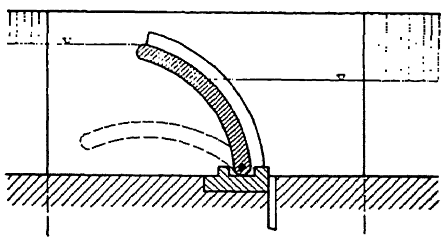


Fig. 240. Blockland weir

A type of structure which may be designated as a curtain weir but which does not roll is the *Bremer Blockland* weir. Its operation is illustrated in Fig. 240. Arrangements were necessary to raise the water level of the turf-ditch net of the *Bremer Blockland* in order to provide a head of about 30 cm. (11.8 in.) without causing a hindrance to the passage of numerous boats. These requirements were fulfilled by the invention of the Blockland weir. Wooden stops are fixed to both side walls and support the trap on the upper water. The wooden trap is riveted to a leather sheet, but remains flexible enough to allow a turf ship to readily press the curtain downward. When a ship comes from

[1] Hilgard, *Handbuch der Ingenieurwissenschaften*, Pt. 3, Vol. 2.

above, the boatman presses the flap downward with the pole and then passes over the weir which is held down by the bottom of the boat until the latter has passed. A boat traveling upstream goes directly over the damming flap. The loss of water during the short time consumed in going over the curtain is small. The weir is an excellent type of construction which may be highly recommended for all similar drainage districts which maintain boat traffic.

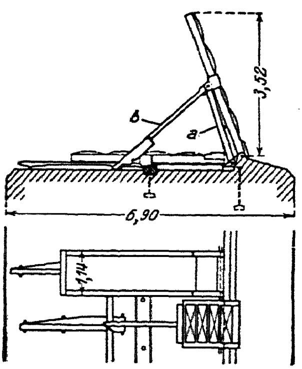
c. Gate Weirs

1. Trap Weirs

a. Trap Weirs Which Are Laid Over by Headwater Pressure but Are Raised Mechanically

In its simplest form a trap-weir arrangement consists of a plate hinged to a fixed bearing on the lower end and held erect by an inclined supporting bar. The support can be moved sideways with a hook to allow the trap to fall. This primitive arrangement is occasionally used for very simple crest weirs, but, in general, it is not to be recommended.

The Chanoin trap weir (Fig. 241) is based upon the foregoing fundamental principle except that two-winged traps with special support are used. Support is provided by two struts, *a* and *b*, of which *a* is hinged at the top and bottom while *b* is connected with a hinge only at the top. Strut *b* is held at the bottom by a narrow counter support. The trap falls over (as sketched in the drawing) when the foot of *b* is pushed sidewise. The displacement is caused by a movable sliding rod which has as many projecting noses as there are traps. The projections are spaced in such a manner that only one trap is tripped at a time. The main improvement in this layout rests in the arrangement of the trap itself.



Figs. 241 a and b.
Traps with collapsible supports

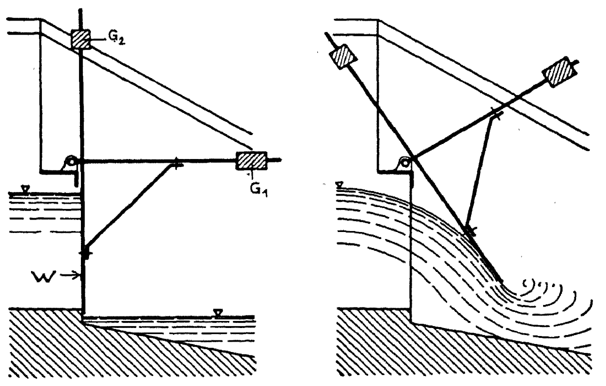
As long as the resultant of the water pressure, W, remains below the turning point of the trap, the trap remains standing, but if the center of pressure rises above this point, the trap folds over without falling all of the way to the floor. It maintains an intermediate position. If the water recedes, the trap can again be brought to an upright position by means of a hook. Furthermore, the trap can be laid over completely. The traps are usually about 1.5 m. (4.9 ft.) wide, the space between these amounting to from 5 to 10 cm. (2 to 4 in.), so that the weirs are not particularly waterproof. The greatest height

of trap thus far used for these weirs is 5.4 m. (17.7 ft.). In order to make the raising of the folded weir more convenient, a needle bridge is set in front of the trap weir. The entire system is very complicated and is not to be recommended. Consequently, many interesting particulars are not discussed in this treatise.

β. *Weir Traps Which Are Opened by Water Pressure and Closed by Counterweights*

a. The Doell Trap

This is a completely automatic trap proposed by Doell (Figs. 242 a and b). The water pressure, W, is held in equilibrium by the counterweight, G_1. When the headwater rises, the pressure, W, increases and turns the trap upward. The lever arm of G_1 becomes shorter as the trap turns, causing the latter to fall over, provided W does not decrease too greatly when the water begins flowing. There is a corresponding condition of equilibrium for every headwater stage. The weight, G_2, merely serves to balance the weight of the lower half of the trap. This arrangement is principally of theoretical interest. It is very difficult to maintain a definite water stage with such traps. However, they are quite convenient for providing a definite high-water limit. The arrangement is usable for the closure of narrow channels.



Figs. 242 a and b. Automatic traps by Doell with counterweight above trap

b. Crest Weirs

Crest weirs are at present much used for raising the crest of fixed weirs. They are arranged to operate entirely automatically. The traps are expected to hold the upstream water-level approximately constant until they are completely laid over. The fixed weir height is then reached and the traps are no longer effective. There are a large number of such types of construction, five of which are here described.

Crest Weirs with Underhung Lever Counterweight

This is a trap hinge connected at the bottom (Figs. 243 and 244) and provided with a counterweight which hangs in a cavity under the fixed weir. The counterweight hangs on the opposite side of the fulcrum from the trap and thereby holds it erect. As the water stage rises, the trap is forced down a corresponding amount, whereupon the center of the water pressure on the plate rises and the lever arm of the counterweight becomes longer. The dimensions can be so chosen that the headwater stage remains approximately constant until the trap is completely laid over. The hollowed-out space under the weir may be closed off and heated, in order to decrease the danger of freezing the trap. Likewise, the side supports of the trap may be heated as a safeguard against freezing. A detail of the bearing at the fulcrum is shown in Fig. 244.

Crest Weirs with Lever Counterweight above Trap

Fig. 245 shows an arrangement which avoids the construction of an underhung counterweight. The difficulties which may arise through the freezing of the counterweight are here avoided. If the lever on which the counterweight hangs is supported on an ordinary bearing, the same possibilities are presented as in the arrangement shown in Fig. 243. It is expedient, however, to allow the lever to bear upon a curved, toothed

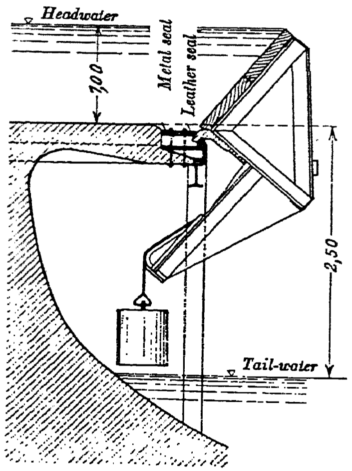


Fig. 243. Automatic damming trap with underslung counterpoise. *Stauwerke A.-G.*, Zurich

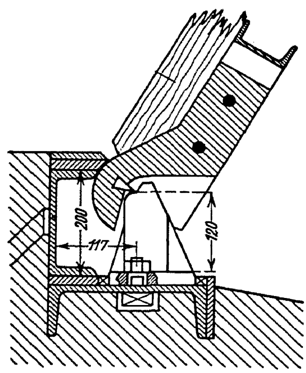


Fig. 244. Fulcrum bearing

disk in such a manner as to attain a weight equalization making it possible to hold the headwater at constant elevation. This condition is also attainable by placing the curved disk at the end of the lever, so that the

cable (on which the counterweight hangs) rolls over the curve. The simplest arrangement is obtained in this way and presents probably the most advantageous type of the various crest gates. Fig. 246 shows such an arrangement. The counterweight is directed by means of a roll so that the lever arm of the counterweight increases in length as the trap is laid over by the increased water pressure. The advantage of having the movable parts above water is also given in this case.

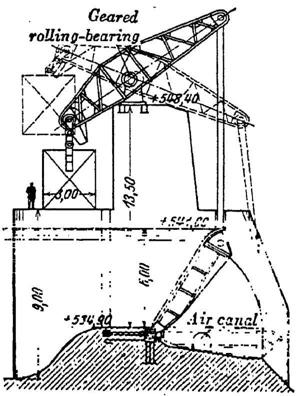


Fig. 245. Damming trap with overhead weight on a lever

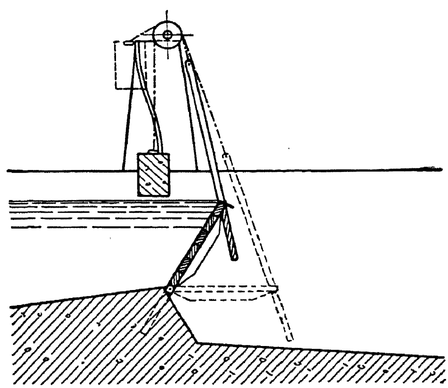


Fig. 246. Crest gate with counterweight on a cable over a curve. *Stauwerke A.-G.*, Zurich

Automatic Crest Gates with Overhead Rolling Counterweight

As shown by Fig. 247, it is possible to arrange a roller over the opening, both ends of which are connected to the trap by a cable, so that an

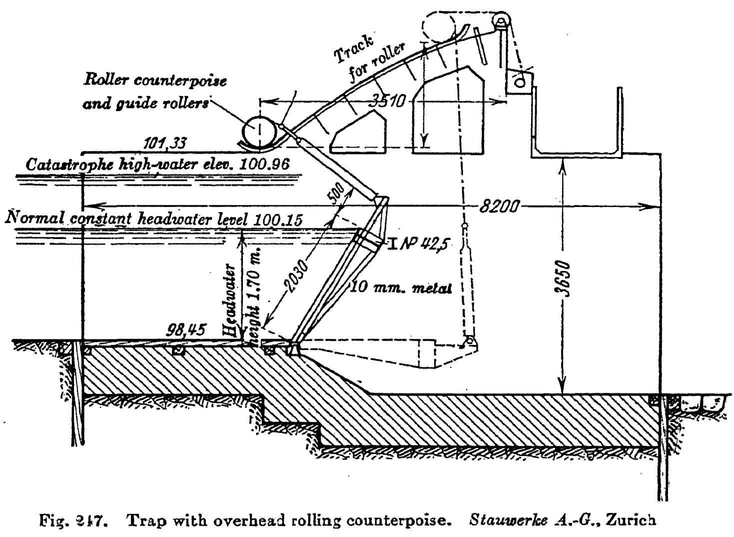
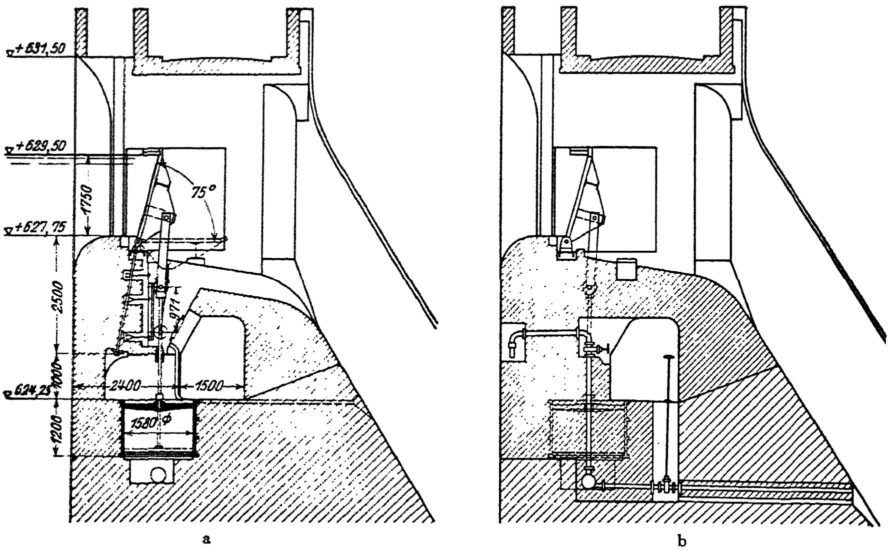


Fig. 247. Trap with overhead rolling counterpoise. *Stauwerke A.-G.*, Zurich

equalization of water pressure can be obtained. By proper design of the roller track, it is possible to obtain an equalization so that the head-water persists at the same elevation until the trap is completely opened. The operating cable is arranged in a manner (Fig. 247) causing the roller to turn in a clockwise direction, rolling upward when the trap lowers.

Crest Weirs Provided with Pressure Pistons

A form of crest gate different from those so far described is actuated automatically to open the spillway of the *Lampmannsperre* (Figs. 248 a and b), a German dam. As shown in Fig. 248a, the piston is under



Figs. 248 a and b. Automatic damming trap of the Lampmann dam. a Section through the lift cylinder b Section through the inflow and discharge conduits used for providing water pressure

pressure from the bottom when the upper valve is open and the bottom one closed. This pressure is sufficient to hold the trap shut under the permissible headwater elevation. If (in this case) the water rises above +629.50, the trap begins to lie over. The trap is completely laid over in case of highest headwater stage. The trap can be moved arbitrarily by opening the lower valve.

γ. *Weir Traps which Operate Entirely Hydraulically*

1. Ordinary Two-arm Traps

Fig. 249 shows the simplest form of entirely hydraulically operated weir. The trap is so supported that up to the water stage at which no

water is to flow off, the force, W, coincides with the point of support. As soon as the water rises further, the force, W, rises above the point of support and the weir folds over. It is hindered from folding over completely by a chain or rebate so that when the headwater drops, the weir again automatically takes an upright position.

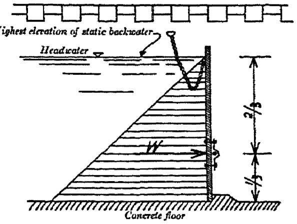


Fig. 249. Automatic swing trap

2. Drum Weirs and Angle-Sluice Weirs

The drum weir is a development of the ordinary trap weir in which the upper portion of the trap over the reaction is used as a dam and the lower part is enclosed in a cylindrical chamber or drum. Fig. 250 shows a drum weir of modern construction. The bearing is hollow and consists of single, cylindrical pieces which rest individually on corbels. The upward projecting plate of the trap girder is constructed between these separate pieces. The sheet-metal surface of the trap scrapes at the top and in the chamber from the bottom on the outer surface of the bearing cylinder. The trap girders are crimped along the lower portion so that there is sufficient space for the cover girders when the trap is laid over, and additional space remains above the tipped gate. The front and back chambers both close water-tight against the channel. The space above the lower trap is connected with the culvert, K_1; the space below with the culvert, K_2; by means of a four-way cock. Two further culverts lead from the four-way cock (Fig. 251) to the headwater and tail-water. It is thereby possible to connect K_1 with the headwater and K_2 with the tail-water or vice versa. In the first case, the entire headwater pressure in the direction of flow acts upon the lower portion of the trap, but the

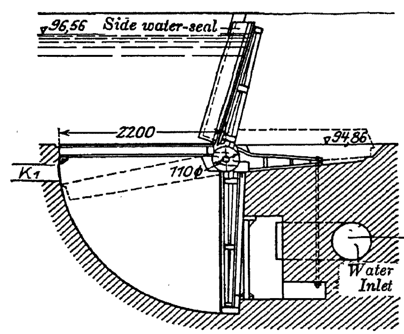


Fig. 250. Drum weir for the pool at Kesselstadt

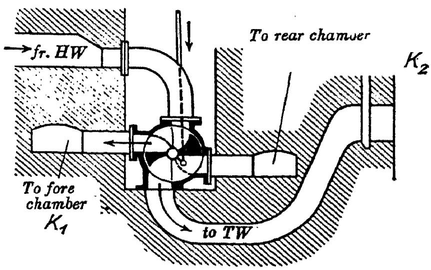


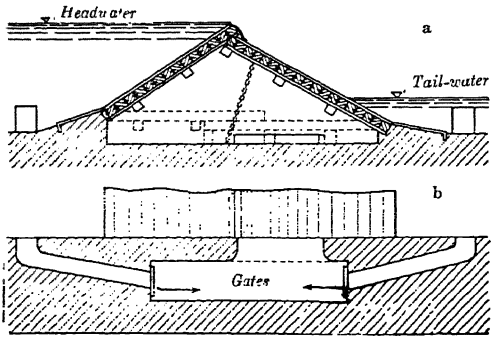
Fig. 251. Means of regulation for a drum weir

pressure in the reversed direction is only the tail-water head. The upper trap is acted upon by the full headwater pressure. The tail-water pressure on the lower trap is about three times as great as the headwater

pressure on the upper trap. The moment holding the trap erect is therefore considerably the greater. The length of the lower trap should be designed to provide for sufficient overweight. If the four-way cock is turned, the headwater pressure acts as a pressure trapezoid on the tail-water side of the lower trap, causing the weir to be turned over. The tipping is abrupt and results in impact. These weirs are much used for closing raftways. At the time when the use of electricity was still in its infancy, the automatic operation was considered a great advantage. As the structures are comparatively expensive, it is now usually preferable to install simple traps, segment weirs, cylinder weirs, or the like.

3. Automatic Double-trap Weirs

Fig. 252 shows this type of weir in its original form. Two traps are supported over a channel forming a roof-shaped structure so that the headwater trap projects over the tail-water trap. The hollow space below may be connected with either the headwater or the tail-water by means of a side culvert which contains sluice gates. If connection is made with the headwater, the pressure on the headwater trap is practically the same inside and outside. The inside pressure on the tail-water trap then corresponds to the headwater pressure. The tail-water trap is the actual lifting apparatus. If the gate openings in the canal are reversed, water flows out of the weir to the tail-water. The head trap receives a surcharge and presses the lower trap down to a horizontal position. The headwater trap thus is the apparatus forcing the gate open. A chain fastened from the tail-water trap to the bottom of the channel hinders the structure from springing past the end position. When the traps are made of wood having a specific gravity approximately equal to that of water, a few centimeters of headwater pressure suffices to raise the weir. If the traps are heavy, for example, of steel or reinforced concrete, then a c erta amount of overpressure must be created before the double trap can be raised. These weirs are frequently called "bear-trap" weirs, a nickname which was given them by the workers who constructed the first weir of this type on the Ohio River.



Figs. 252 a and b. Automatic double-trap weir

Special attention is given to providing an impervious joint between the traps. They may either be allowed to scrape each other, or the two traps may be tied to each other by a hinged joint; in the latter case, one of the two must scrape on the floor. In using two traps, one gliding joint is unavoidable; a particularly tight connection cannot be attained along this joint and there is continual danger of sand washing into the chamber. Hence, special cover traps have been applied; also one trap has been resolved into two so that instead of a gliding joint, a collapsible joint is provided. Fig. 253 shows a double-trap weir with cover trap. Fig. 254 indicates the resolution of the tail-water trap into a double trap tied together by hinges, an additional cover trap being installed. The same measure has also been applied to the head trap. A more suitable arrangement results in the latter case than with resolution of the tail trap because of the pressure equalization on both sides during the damming. According to Hilgard, these weirs have proven themselves excellent. Several dozen have already been constructed. In the United States, they are considered a particularly good type of construction. In Germany, they have been used very little, although they appear to be very suitable because they can readily be made to allow ice to pass. Different weir heights can be obtained by various positions of the sluice gate. Nevertheless, they are not suited for very fine regulation of water stages and probably can not be made exceptionally water-tight.

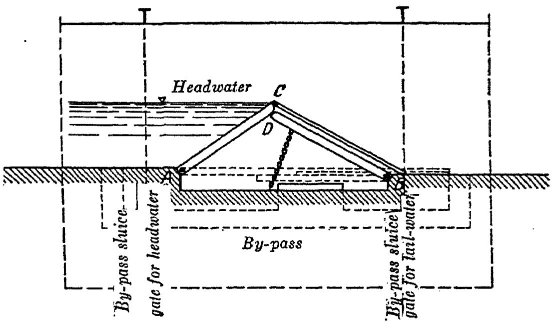


Fig. 253. Automatic double-trap weir with cover trap for protection from detritus

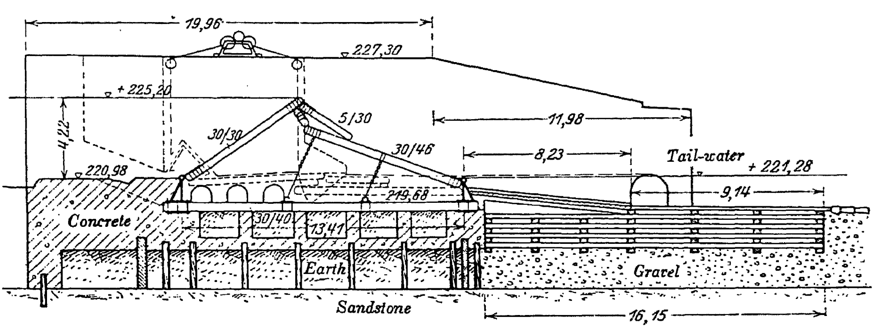
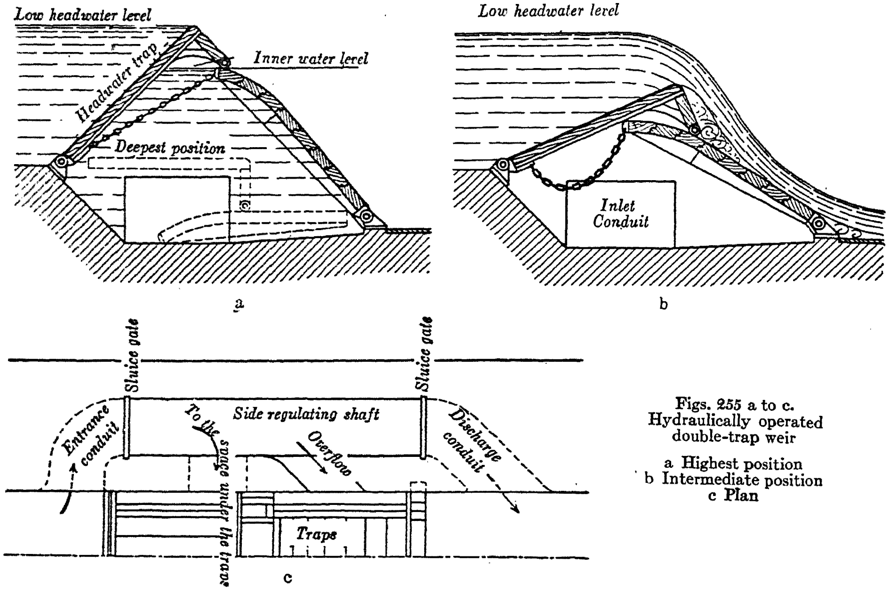


Fig. 254. Double trap on the Chicago Drainage Canal

A special design of the double-trap weir was developed in Switzerland (Figs. 255 a to c). Here the head trap is roof-shaped so that the



Figs. 255 a to c. Hydraulically operated double-trap weir
a Highest position
b Intermediate position
c Plan

whole structure appears as a roof. This type can be made more nearly waterproof than the ordinary design. These weirs are now frequently constructed in Switzerland. The disadvantage of difficult accessibility to the lower bearing has not been remedied in these structures.

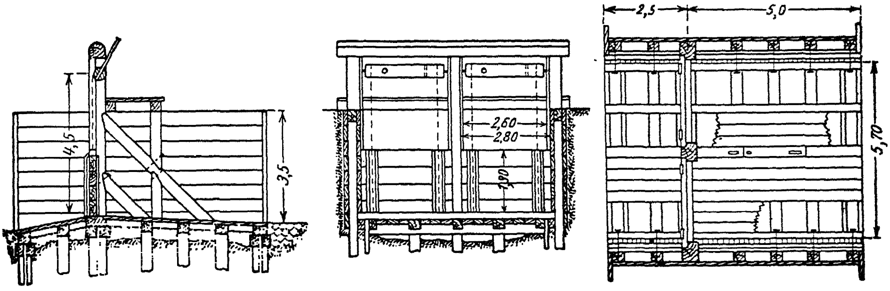
The broadest, continuous, double-trap weir section has a width of 48.6 m. (159.5 ft.) and a head of 5 m. (16.4 ft.). For very great widths, transverse bending may occur, because the pressure reaches its full value first in the vicinity of the pier and requires a short time to spread to the center. The traps are, therefore, stressed more at the sides than in the middle in raising; the reverse is true when emptying. These disadvantages can be reduced by special arrangement of the culverts.

2. Sluice Weirs

a. Simple Sluice Weirs for Mill Channels and the Like

Sluices as closing arrangements for mill channels have been known since ancient times. It is probable that the ancient races already used similar closures in their great irrigation projects. The simplest sluice gates consist of wooden planks tied together by iron bands. These gates slide in grooves. In opening the sluice, a large frictional resistance, usually greater than half the water pressure, must be overcome. It was not until recent times that fixed rollers and roller bearings have been

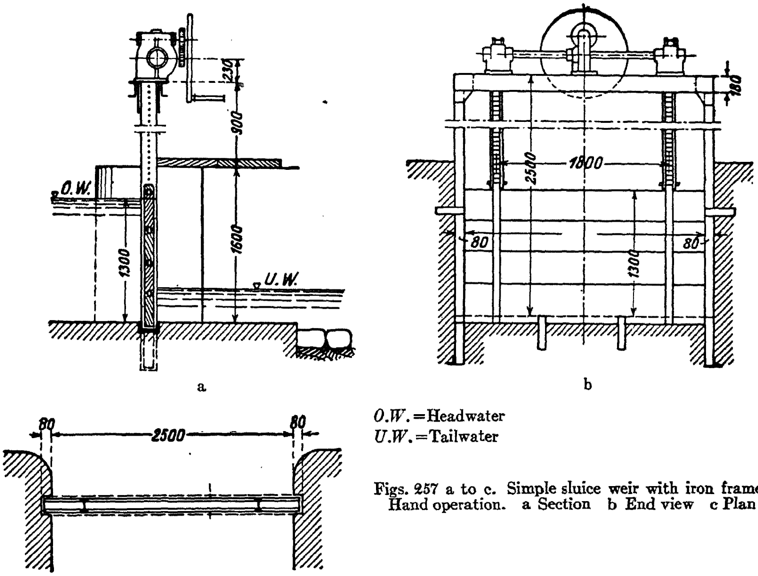
used to diminish the friction. Figs. 256 a to c show a simple mill-sluice similar to those used at the present time. In order to make it possible to



a b c

Figs. 256 a to c. Mill sluice gate. a Cross-section b End view c Plan

close off wide channels, either fixed or removable intermediate supports have been erected so that the opening is divided into several parts. The removable uprights used to be made purely of wood construction. Sluice gates are, at present, invariably provided with rigid rods for opening



a b c

$O.W.$ = Headwater
$U.W.$ = Tailwater

Figs. 257 a to c. Simple sluice weir with iron frames. Hand operation. a Section b End view c Plan

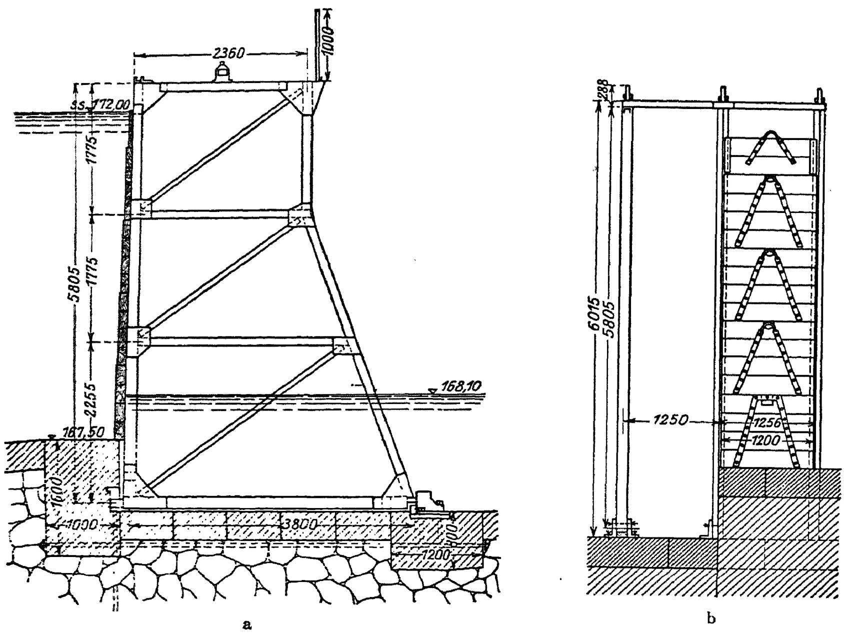
them, since these are necessary to force the gate down. A very simple arrangement of this type is given in Figs. 257 a to c, which shows a new construction type having toothed rods and with the plate sliding in

channel irons. Screw spindles have also been used for operating such sluices. In order to reduce the frictional resistance, the sliding areas are often lined with bronze plates.

Formerly when a high-head arrangement was necessary, the sluices were also divided into several sections vertically, the gates being withdrawn one after another. The resulting layouts were therefore somewhat complicated, especially where movable standards were used. All such designs are now out of date and should be used only for simple layouts, when only one large gate should be installed, preferably resting against rollers.

β. *Newer Weirs with Movable Intermediate Supports*

The old type sluice weirs with movable wooden piers pointed the way for the introduction of the newer weirs with intermediate supports. In order to close large openings, and also to permit opening them completely, it was necessary either to make intermediate supports similar to the trusses for needle weirs, or to span the entire gap with a single steel



Figs. 258 a and b. Sluice weir at Liebschitz
a Elevation of a truss b Front view of one of the gates

bridge against which the top of the uprights could rest, the lower support being the base of the weir. Weirs of the latter type are best designated by the name bridge weirs.

1. Sluice Weirs with Intermediate Trusses

Sluice weirs with movable intermediate trusses are used at the present time. Figs. 258 a and b shows such a weir as constructed at Liebschitz in connection with the Moldau canalization system to allow for a ship passageway of 65 m. (210 ft.) clear width. It requires ten laborers a period of six hours to completely open the ship passageway; only a few minutes is required to free the same opening by means of one or two segment weirs.

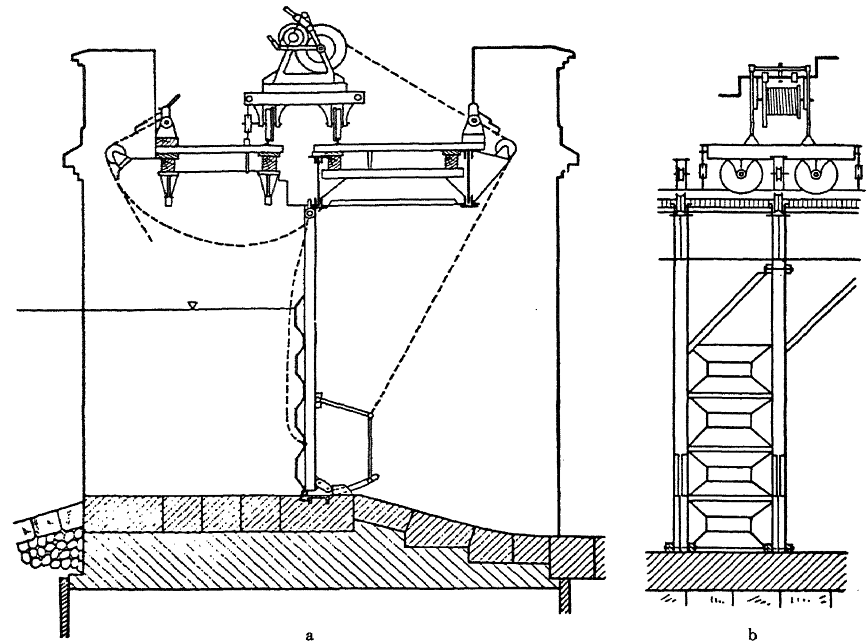
The diagrams represent the weir system clearly enough to make detailed description unnecessary. Movable windlasses are placed at disposal for changing the sluice gates. The trusses are coupled in groups of six which may be laid over or erected by a windlass fixed to the pier. The entire layout is very complicated, delicate and uneconomical. It is a bad type of construction compared with the newer segment and cylinder weirs. Its greatest disadvantage is that it is not suitable for by-passing ice by lowering of the weir.

2. Bridge Weirs

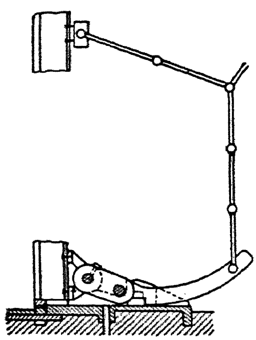
The oldest weir of this type is the Pretziener weir which serves for closing the by-pass canal of the Elbe above Magdeburg up to a definite high-water stage, and is opened only as high-water relief. In the Pretziener weir (Figs. 259 a to c) the tops of the movable standards are hinge supported on the bridge, and the bottoms bear on the weir slab in a manner to permit uncoupling them. The Pretziener weir was a pioneer and is therefore discussed because of its historic significance. The weir has a total length of opening of 162.8 m. (543 ft.) with nine openings, each of which has 8 removable standards. Small sluice plates of corrugated sheet iron .84 m. (2.75 ft.) high and 1.31 m. (4.3 ft.) wide are placed four high in vertical rows; they are pulled by cables. The bottom bearing is uncoupled by a lever; thus, after lifting out the sluice plates and unbolting the standards, the latter can be flapped out in the direction of the river current. The latter arrangement is necessary because of floating ice. The weir is out of date as to construction, but this type of structure has been installed at several other places, better construction being used in the more recent projects. Earlier developments, in which the hinge of the loose standard laid on the bottom of the weir so that the standard was laid over in the bed, are unsuitable because of the difficulty with movable parts under water and because the standards become covered by detritus.

Weirs similar to the Pretziener have been constructed in Austria. In one of these, which is at Mirowitz in the Moldau River, a fixed rebate was used for the loose standards, making it necessary to fold the stand-

ards back against the stream. During times of floating ice, this arrangement is dangerous and should be used only in rivers where it is certain that floating ice will occur only at times when the weir is open, which is very infrequent.



a b



c

Figs. 259 a to c. Pretziener weir
a Cross-section
b Elevation from headwater side
c Catch arrangement for the loose standards

Improved forms of the Pretziener design have been built in the Mulde at Bitterfeldt, in the Weser at Doerverden, near Breslau in the Oder, and other places, however, without getting rid of the fundamental limitations of the structure; namely, that it is too sensitive, that it can not pass ice by lowering the weir, and so on. Unfortunately on the Weser, Oder, and other rivers, the old system of sluice weirs with many standards has been retained even for new structures, despite the fact that better weir types

have been invented, such as cylinders, segments, and sectors. The weir at Mirowitz in the Moldau is at least a step forward, in so far as here, instead of separate sluice gates one above the other, a single high sluice gate of 5.3 m. (17.4 ft.) height and 1.84 m. (6.04 ft.) width was installed. These plates lay on roller trains. Fixed rollers would have been better. Instead of having two sluice gates supported at both sides of one standard (Fig. 260) consisting of an I-beam, two channels were used independently of each other, and the two channels serving to guide one plate were connected with each other (Fig. 261). Each sluice gate is thereby independent of the adjoining gate. In addition, it is not necessary to take the sluice plates out of the guide standards; they remain hanging in the upper part of the standard and are folded over with the standard. Another notable feature is the well-developed bridge support for the standards. In spite of the excellent detail arrangement of this weir, however, it cannot stand the competition of cylindrical weirs and should not be built again. Large sluice weirs without intermediate supports are also superior to this type.

Fig. 260

Fig. 261

Figs. 260 and 261. Old and new arrangements for bearings for sluice gates

Lift-bridge weirs, such as used at Melnik in the canalization of the middle Elbe and in the New York barge canal, form a special group. The design follows an entirely individual method in that the bridge is lowered to a level only slightly above the highest head, but is transformed into a lift bridge. The entire system of standards naturally becomes much lighter because the standards are much shorter. They are loosened by lifting the bridge a few decimeters, thus causing their support at the lower bearing to be disengaged, and can then be folded back, the bridge being raised high enough to free the entire opening. In any case, this is the best type of sluice weir with movable standards. A similar effect was obtained at Breslau by arranging a heavy eccentric which raises the standards above the sill. A particular type has been developed for the closure of lock canals including those at Sault Sainte Marie in America and at the Panama Canal. Here the swing bridge weirs accomplish a very similar effect to that of the lift bridges. For sea ships, because of the necessity of high lifts, lift bridges were uneconomical.[1]

γ. *Newer Weirs without Movable Intermediate Supports*

1. General

Sliding sluice gates are generally not usable for covering wide openings because the frictional resistance becomes too large.

[1] O. Franzius, Der Panamakanal, *Z. V. d. I.* 1914.

Arrangements such as those in Wangen on the Arre River, in which sliding sluices of 4.6 m. (15.1 ft.) width and 2.1 m. (6.9 ft.) height were used, may be considered out of date. The windlasses for raising the gates must be made too heavy because of large forces to be exerted. The example of Wangen shows that these difficulties can be overcome, but it does not verify the correctness of the design.

Roller sluice gates can be actuated between stone piers and also between iron standards. Since the gates are usually broad, restrained guidance is very important, because otherwise they might easily become jammed. Similarly, a waterproof packing at the sides of the gates is especially important since the rollers require a small intermediate space between the sluice gate and the pier. This intermediate space must be bridged over by some sort of sliding water-seal without adversely affecting the movement of the rollers.

Stress analyses for sluice gates are based on the assumption that the gate is a beam resting unrestrained on two supports. For large gates, either plate girder or lattice girder construction is necessary. The plates are joined together with horizontal beams tied together by transverse vertical members, at the headwater side of which a sheet metal cover is riveted. The work is cheapened if the horizontal beams are of equal strength. This is attainable up to certain limits, by the following method (Fig. 262). Above the tail-water level, the water pressure increases from top to bottom as a triangle; below the tail-water, the pressure distribution is rectangular. Thus below the tail-water the pressure is uniformly distributed.

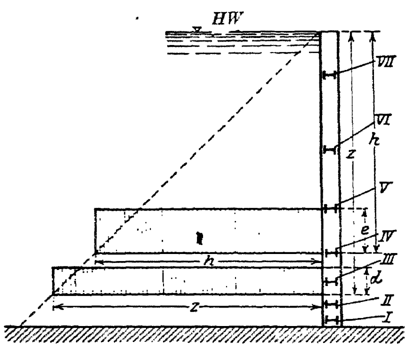


Fig. 262. Computations for the design of a steel sluice gate

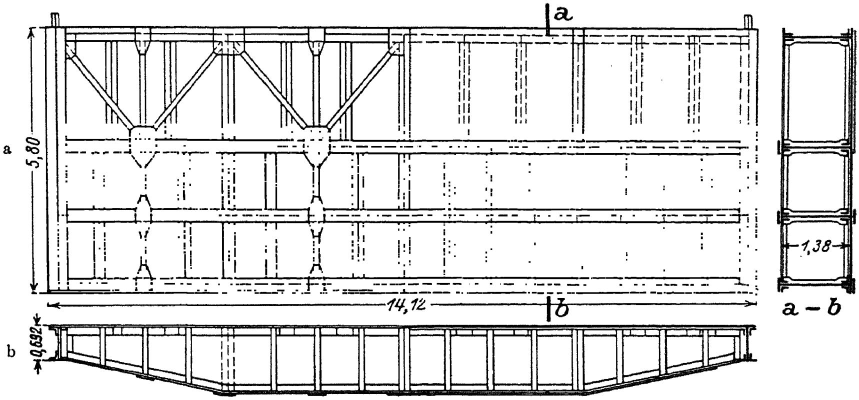
The unit pressure of the water above the tail-water level for a head H, is $W = \frac{H^2}{2}$. It must be decided how much pressure one horizontal beam is to support. For example, if one beam is to support $\frac{1}{n} W$, then each area must be made large enough so that the pressure on the area amounts to $\frac{1}{n} W$. For the strip below the tail-water level the interval is

$$h_n = \frac{H}{2n}, \text{ then } W_n = \frac{H^2}{2n} = \frac{W}{n}.$$

The height of the top triangle is $h_1 = H\sqrt{\frac{1}{n}}$, the total height h_2 of the second triangle is $h_2 = H\sqrt{\frac{2}{n}}$, the third $h_3 = H\sqrt{\frac{3}{n}}$, etc. From this it follows that the triangular areas are

$$\Delta_1 = \frac{H^2}{2}\frac{1}{n} = \frac{w}{n};\ \Delta_2 = \frac{H^2}{2}\frac{2}{n} = \frac{2}{n}w;\ \Delta_3 = \frac{H^2}{2}\frac{3}{n} = \frac{3}{n}w.$$

The difference from triangle to triangle is thus always $\frac{w}{n}$. Hence each



Figs. 263 a to c. Stony sluices on the Gatun dam
a Elevation b Horizontal section c Vertical section a-b

strip has the same pressure to support as is given by the area of the top triangle. The intervals can readily be computed with a slide rule. If the tail-water can become very shallow, it is recommended to neglect it entirely because, as a result of the hydraulic jump arising when the weir is raised slightly, the tail-water may become practically zero.

Figs. 263 to 265 show diagrams of large sluice gates which have been erected. The structures are of standard steel construction. It is particularly important to avoid hollow places which can not readily be painted. As in lock gates, it is better to avoid curved metal and to make sharp bends. Crimped angles are unavoidable but are not harmful. Diagonal stiffeners are usually dispensable since the cover plate forms sufficient stiffening.

Sluice gates, in consideration of convenient construction, are usually supported in the grooves on areas which lie parallel to the gate. This bearing does not take into consideration stress due to temperature expansion. High stresses may be developed in the sluice gate as a result of variations in length due to temperature changes. The normal

reaction may be forced out of line as much as 30° or more. When the gate shrinks due to cooling, the abutment pressure is directed outward toward the free side. Shearing off of the vertical groove can then easily occur (Fig. 266), if adequate provision is not made by use of heavy iron inlay. This danger can be avoided by constructing the bearing areas at an oblique angle to the pier, a procedure proposed by Eggenschwyler. It has the added advantage of weakening the pier less than the rectangular grooves.

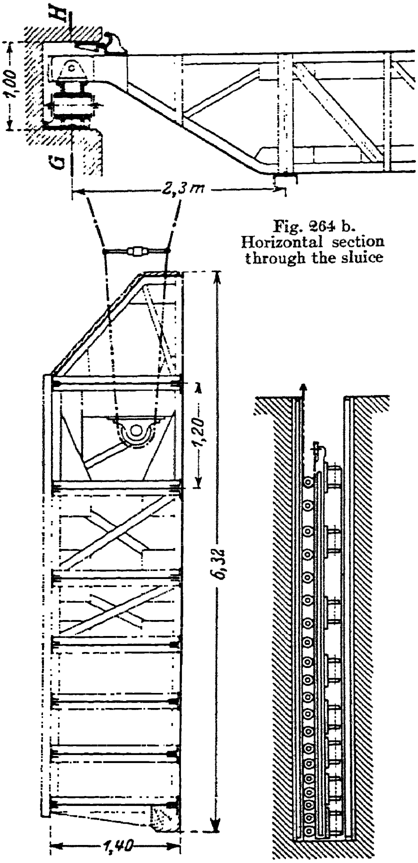


Fig. 264 b. Horizontal section through the sluice

Fig. 264 a Vertical section

Fig. 265

Figs. 264–265. Sluice weirs on the Aare in the Beznau

Fig. 265. Vertical section G–H through the bearing and the roller train

2. Roller-Bearing Supports and Water-Seals

Rollers used for sluice gates are divided into two groups, namely fixed rollers and roller trains. Roller bearing supports in themselves present but little that is new, since these have been extensively developed in connection with machine design. The water-seal, on the other hand, is quite important. Figs. 268 and 269 show bearings on fixed rollers. The first figure shows a sluice gate made of wooden beams, the water-seal being spring steel. The second arrangement (Fig. 269) shows the sluice gate in the by-pass of a lock in section. The seal in this case may be the same as for weirs, since there is no fundamental difference. Here, also, for the ordinary roller gate a side water-seal can most readily be obtained by the use of pieces of elastic sheet-metal. Gates have been supported on fixed rollers (by Louis Eilers) for pressure tunnels of dams, with water pressures of several hundred meters.

In the case of large gates in which a movement by the wind is to be feared after the gate has been pulled from the water, a counter-roller on the opposite side is also necessary. It is advisable to place

the water-seal on the upstream side, if possible, so that the movable parts are on the air side. Painting can then be readily accomplished during low-water stages.

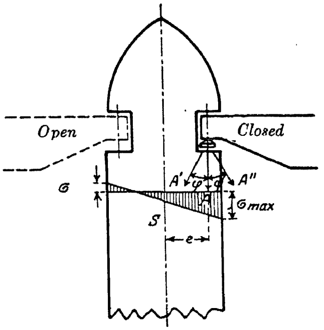


Fig. 266. Bearing at right angles to pier

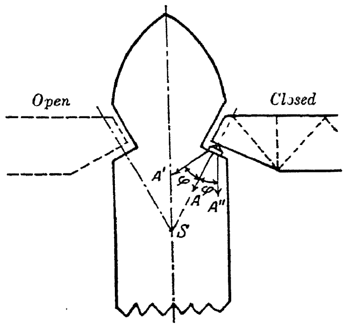


Fig. 267. Bearing inclined to pier

Figs. 266 and 267. Stresses incurred in piers supporting sluice gates

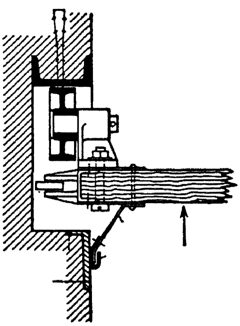


Fig. 268. Fixed roller bearing

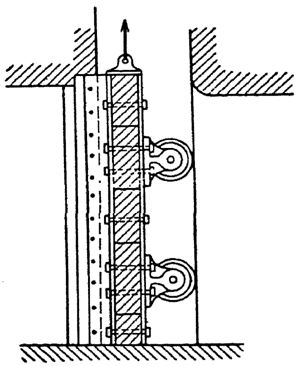


Fig. 269. Fixed roller bearing

Roller-bearing trains (Figs. 265 and 270 to 273) are disadvantageous in that the roller trains move only half as fast as the gate. The roller train must be hung on the gate, because otherwise, after the gate is completely raised, the rollers would fall out. The trains are hung from a simple pulley block (Fig. 275); the hanging roller need not be of the same diameter as the bearing rollers. One end of the hanging cable is fastened to the gate and the other above on the pier.

A suitable method for rapidly lifting the roller train after raising the gate from the water was devised for the sluice gates of the open pass of Gatun Lake at the Panama Canal (Figs. 276 a and b). Here a manifold block and tackle was inserted into one cable, the two roller settings of which were kept from approaching each other by a header. As soon as the gate is raised over the water, it is pressed away from the support by a roller so that the roller train hangs free; a hook which is fastened to the pier then extends into the lower pulley and holds it fast. The pulley block now pulls itself apart, and thereby raises the roller train at a mul-

tiple velocity so that at the end of the travel of the sluice gate, the train no longer hangs down below the gate.

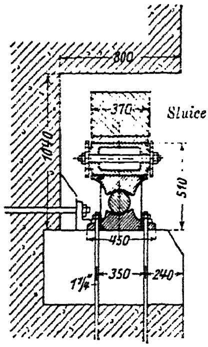


Fig. 270

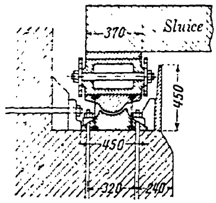


Fig. 271

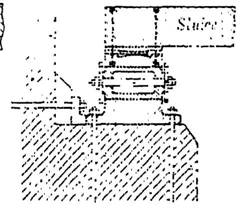


Fig. 272

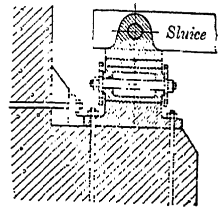


Fig. 273

Figs. 270–273. Side bearings of roller sluices

The bearing on roller trains must be arranged so that the sluice gates are free to undergo flexural deformation without being supported merely on the edges of the rollers. A bearing which is movable transversely must therefore be provided. Provision of this nature was made for the weir at Chevres (Fig. 274). Here the bearing seat on the gate is a hinge connected to the gate. Fig. 274 also shows a method of waterproofing which is frequently used. Between the corner reinforcing of the pier and the Z-shaped member, a round ashwood bar is inserted, forming a good elastic water seal. Experience with this weir showed that water which transports a great deal of fine sand soon causes serious erosion of the roller train if it projects from below the gate when the weir is partly raised. Shields *A* and *B* were installed above and below in order to reduce the amount of wear.

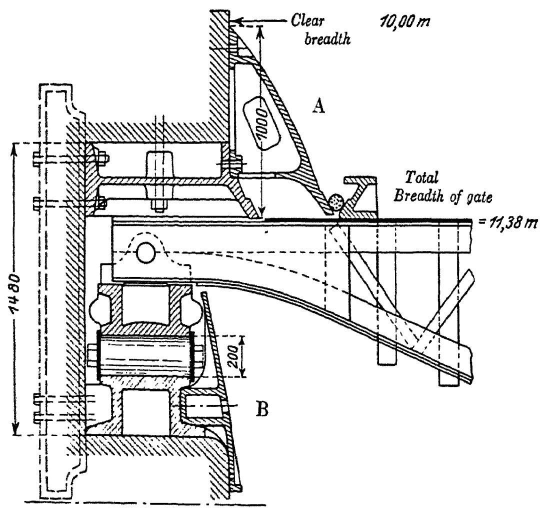


Fig. 274. Roller sluice at Chevres
(protection shields A and B against sand added subsequently)

A particularly good form of sluice gate was developed for the Gatun weir. The water-seal is provided by a strip which is forced against the joint by a spring, and further by a plate which slides along a groove in the wall. Inasmuch as only a limited amount of lateral movement is permissible in this case, an arrangement of counter-rollers and also transverse rollers became necessary in addition to roller bearings in order to prevent jamming of the gate.

The gate is balanced by counterpoises, so its weight need not be lifted; only enough power must be supplied to

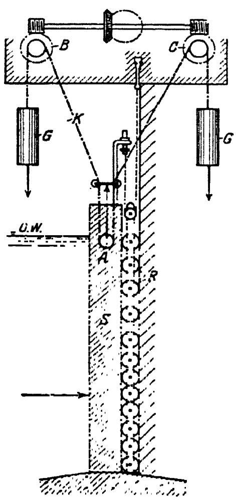
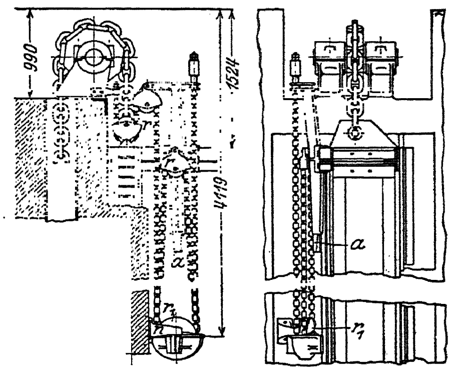


Fig. 275. Roller-sluice counterpoise arrangement



Figs. 276 a and b. Roller train for the sluice gates on the Gatun dam

overcome the frictional resistance of the rollers on which the counterpoises hang and that of the rollers over which the gate slides. The resistance of the upper rollers is very small inasmuch as they are very large and kept well oiled. The resistance of the rollers upon which the sluice gate travels, according to experiments made in the shop with this kind of bearing, amounts to .01 of the load. Experience with the actual structure, however, indicates the frictional resistance to be ten times as great as this, or .1 W, and may become as high as sliding friction. The irregularities in the roller track, abrasion of the rollers, clogging with dirt, non-uniform bearing upon the sluice gate, and particularly freezing, are circumstances causing this increase. Since the sliding resistance may amount to at least .5 W, the use of rollers, provided they turn, results in a diminution of the frictional factor to one-fifth of that for sliding. On the other hand, there is no guarantee that the rollers will not become immovable if the gates are not operated for a considerable period of time. This condition occurs repeatedly.

The counterpoises are hung either from cables, link chains, or Gall chains. Operation is completely independent of the counterpoises. The latter may be allowed to rise and lower along an incline within the pier, in which case the shafts should preferably be water-tight. Wire cables are not suitable for actuating broad sluice gates, because using them necessitates further arrangements to cause parallel movement of the gate. On the other hand, if rigid operating rods are used, such as racks, spindles, or Gall chains, parallel movement may be readily achieved by coupling the raising apparatus to the operating machinery by means of disk or worm gears; for example, the driving pinion need simply be connected to a (preferably rapidly rotating) shaft in order to provide forced operation.

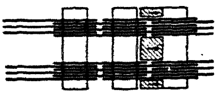


Fig. 277. Patent chain of the M.A.N.

The counterweight cables are joined to the extreme ends of the sluice gate, while the connecting rods in many designs are joined to the gate about one-fourth the distance from the ends. For example, in the Herbrunner weir in the Ems River, a sluice gate of 8.5 m. (28 ft.) length and 2.5 m. (8.2 ft.) height was arranged in this manner. There is a higher degree of accuracy in the displacement if the connecting rods are joined to the ends of the sluice. Such an arrangement supplied with link chains was designed by the M. A. N. Company of Germany, and is shown in Fig. 282.

3. Double-Sluice Gates

The requirement that the top portion of the weir be arranged so that it could be lowered to allow the discharge of ice is seldom fulfilled by the sluice gates thus far described. The sluice weirs previously described, therefore, may be considered largely out of date. The error can be overcome by using double sluices which can be moved past each other. The weir in the Rhine at Lauffenburg presents an old example of such sluices.

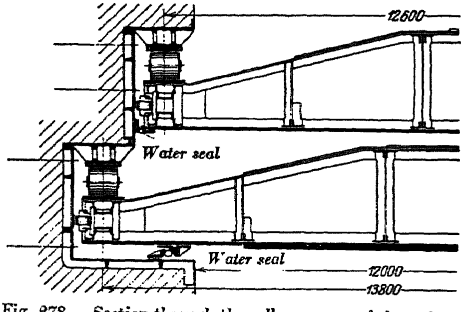


Fig. 278. Section through the roller grooves of the weir at Lauffenburg

In this case each sluice gate is operated independently on roller trains moving in independent grooves (Fig. 278). In this arrangement, the distance between bearings on the lower gate is increased. The water-seal between the two sluice gates provides considerable difficulty. These difficulties will not be discussed in more

detail inasmuch as more recently better types of construction with fixed rollers have been devised.

Fig. 278 shows the manner of providing the water-seal for individual gates. An excellent solution for the water-seal of double-sluice gates has been devised by the M. A. N. Company. Fig. 279 shows the layout for an older type of double sluice gate construction; Fig. 281 shows the new M. A. N. sluice gate. The advantage which the manufacturers stress particularly in regard to their gate is that the lower sluice need not lift the water ballast as in the older type, and also that suction is not in-

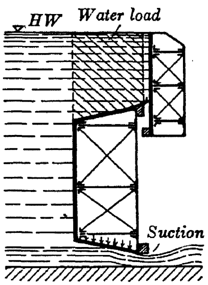


Fig. 279

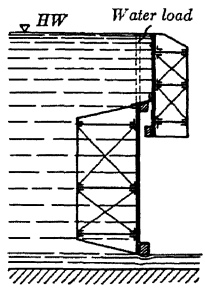


Fig. 280

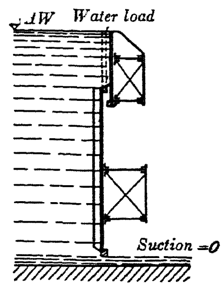


Fig. 281

Figs. 279–281. Suction effect incurred by double sluices

Fig. 279. Usual double sluice, with large pressure and suction action. (The advantage of this type is that the supporting structure is not submerged in water.)
Fig. 280. Usual double sluice, with no pressure and suction action but with the disadvantage that the supporting structure of the lower sluice is submerged in water
Fig. 281. M.A.N. double sluice which combines the advantages of both forms

curred along the bottom of the sluice; the latter condition had the effect of causing considerable additional weight in the older layout. Overcoming the suction provides a great advantage, but the disadvantages of the older designs may be readily eliminated by placing the metal cover on the tail-water side of the lower gate instead of on the headwater side (Fig. 280). The effect is then exactly the same as that of the newer arrangement. The entire supporting structure of the lower gate would be under water, a condition which might be considered disadvantageous. Furthermore, the M.A.N. sluice gate has the advantage that the rollers of both sluices may run on the same track, an arrangement to be preferred. The double-sluice system makes it possible to allow either the top sluice to be lowered for the purpose of discharging ice or the lower one to be raised so that detritus can be scoured out. The water-seal may be provided by two strips adjacent to each other. The joint can be made practically water-tight by adding a leather cover. The M.A.N. Company uses a movable, sheet-metal water-seal. Fig. 282b shows the arrangement of the seal for a sluice of this nature. Consider-

able simplification is brought about in this case in that the two sealing plates are of equal length and adjacent to each other. Another water-seal is required on the wall. The manner in which the plates fit over

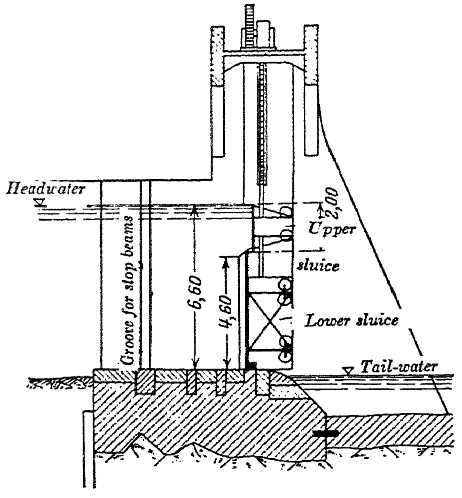


Fig. 282a. Double sluice gate according to a design by the M.A.N.

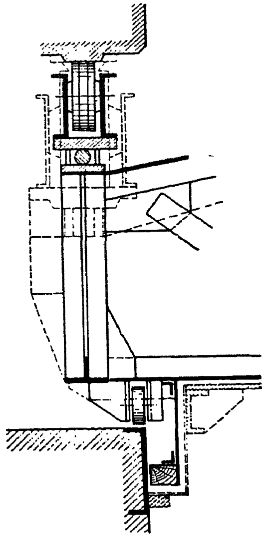


Fig. 282b. The M.A.N. water-seal for a double-sluice gate. Plan.

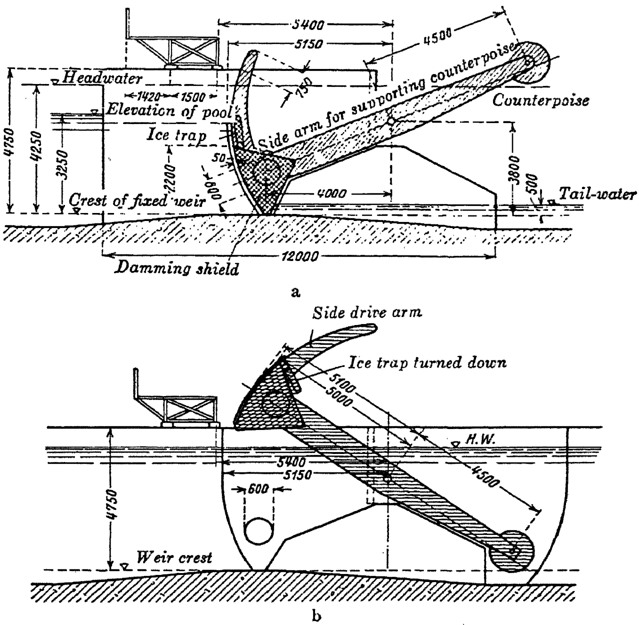
each other is also shown. The provision for a water-tight joint is one of the most difficult parts of the design. The M.A.N. sluices have been built in widths up to 17 m. (56 ft.), and for heads up to 10 m. (33 ft.); however, there is no hindrance to making roller sluices of this nature of still larger dimensions. The latter notwithstanding, roller weirs, sector weirs, or segment weirs are usually preferable to double sluices for large installations.

d. Cylindrical Weirs

1. Segment Weirs

A segment weir consists of a single cylindrical damming shield, supported by individual arms extending downstream from the weir (Figs. 283–288). If the weir is to be opened completely, only two arms are used, one at each end of the gate. These arms are frequently extended backward over the bearing and provided with counterpoises (Fig. 283), so that only frictional resistance in the bearings must be overcome in raising and lowering the gate. Counterpoises may also be arranged to act directly upon the damming shield by connecting them to the shield by means of cables. The most delicate parts of these weirs are the bear-

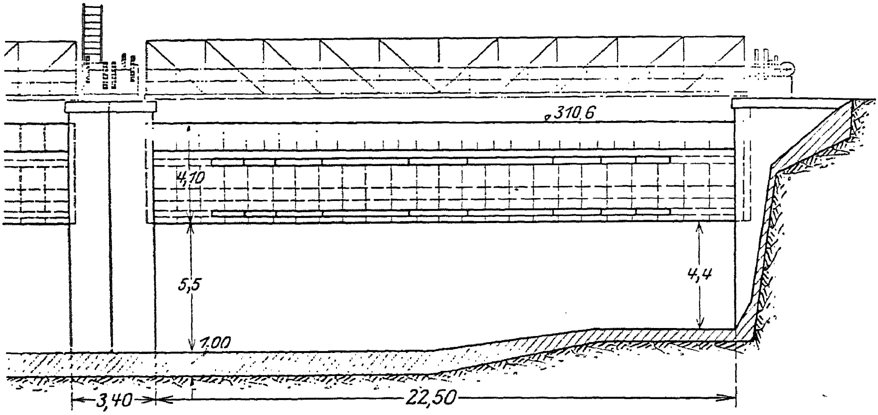
ings, inasmuch as the water pressure must be resisted entirely by them. On the other hand, the concentration of the forces at the bearing points provides great safety, because the design can be made accordingly. The arms rest in rebates which, because of the breadth of the steel construction of the arm, must be deeper than those required for roller sluices. The weirs may be constructed in such a manner that they can be moved upward only or else both upward and downward with respect to the water level. In case they are not built so as to be lowered below the water level, a particular arrangement must be provided allowing the upper part of the segment to be tipped backward. A design of this type is shown in Figs. 283–285. One which may be lowered below the water level is shown in Fig. 288.



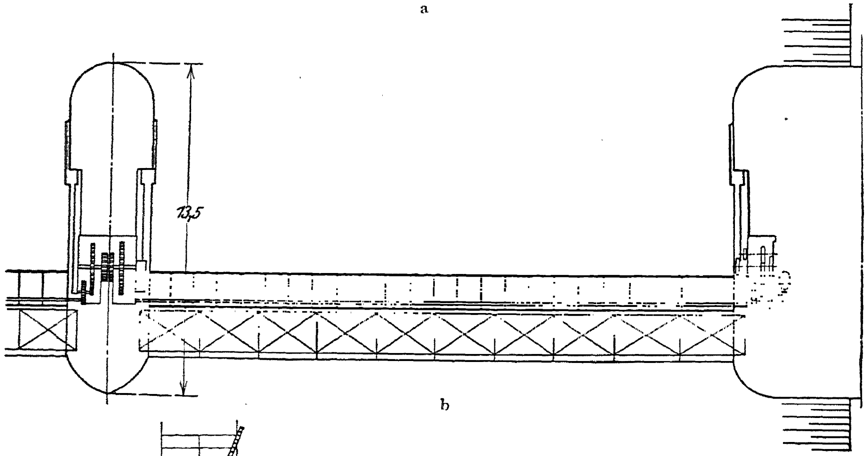
Figs. 283 a and b. Segment weir with counterpoise and ice trap
a In lowered position b In raised position

The framework of the damming wall is computed as a portal or continuous beam over several supports, depending upon the number of supporting arms. Usually the damming wall is computed as a form of box girder. However, since the maximum water pressure is exerted along the lower portion, the principal girders must be located near the bottom of the gate. Thus a hammer-shape form of cross-section results as is indicated in Fig. 283. This form is particularly pronounced for

low damming heights. Inasmuch as there is as much space between the end of the gate and the wall for these gates as in the case of roller sluices,



a



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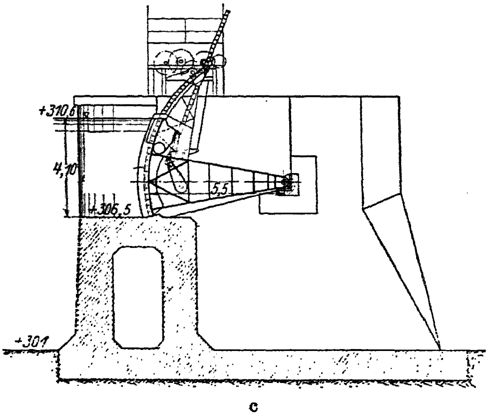


c

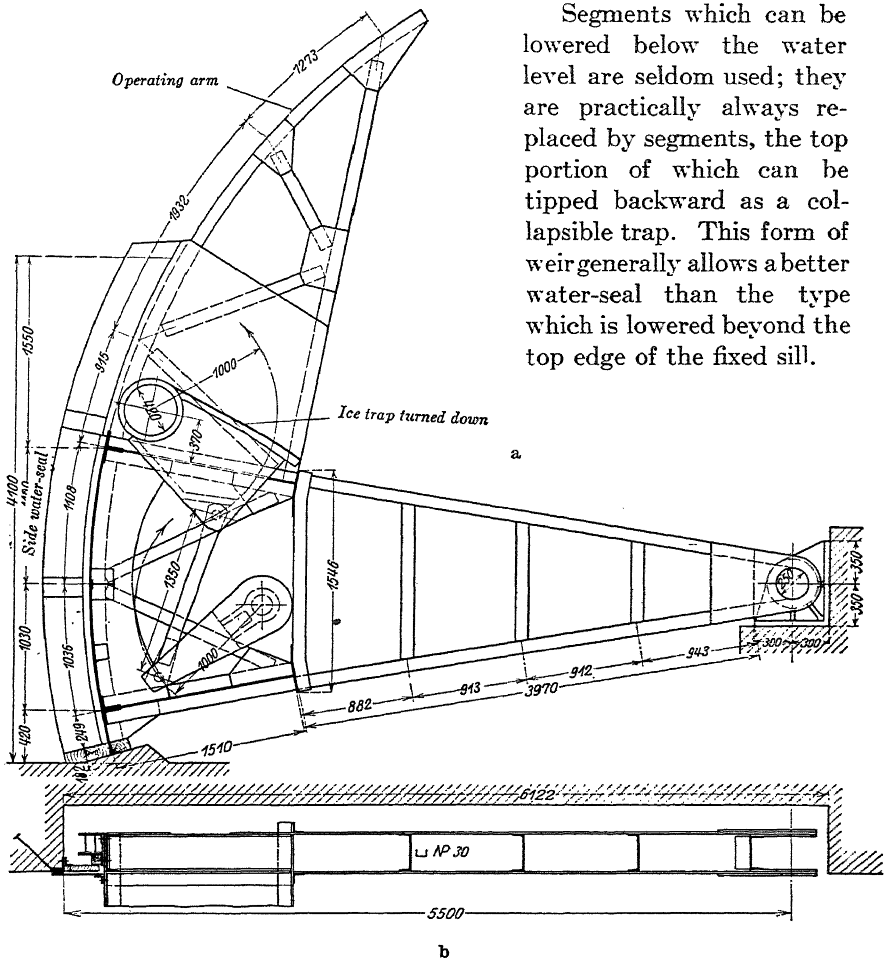
Figs. 284 a to c
Segment weir (Louis Eilers) near Boberroehrsdorf
a Longitudinal section b Plan
c Cross-section

a firm water-seal becomes necessary. The arrangement of the seal does not cause any particular difficulty because the cylindrical damming wall is moved parallel along a cylindrical groove. Spring sheet-metal

surrounded by chrome leather may be used for these water-seals (Fig. 285).

If it is desired to have the weir arranged so that the upper edge can be lowered below the water level, a spring metal strip is also required at the top edge of the fixed sill. The sector weir at Bremen (Fig. 289) has been provided with a good water-seal of this type.

Segments which can be lowered below the water level are seldom used; they are practically always replaced by segments, the top portion of which can be tipped backward as a collapsible trap. This form of weir generally allows a better water-seal than the type which is lowered beyond the top edge of the fixed sill.



Figs. 285 a and b. Segment weir at Boberroehrsdorf
a Section through one of the main trusses. The lowered ice trap lies back of the truss and is therefore not seen
b Section through truss

In making calculations for a segment supported by two arms, the flexural stresses occurring in the arms must be taken into consideration

inasmuch as the damming wall is fixed to the arms. In practice, the bearings are designed strong enough to withstand the stresses incurred as a result of flexure in the arms.[1]

Segment weirs can be constructed in such a manner that the headwater level is automatically held at constant elevation; that is, the weir is arranged to lower automatically when the headwater rises and to rise when the elevation of the headwater decreases. A design of this nature was proposed by Louis Eilers. Segment weirs are actuated either by means of gears at the bearing or by means of a tension rod hinged to the damming shield; the latter is preferable. Nowadays the gate is frequently propelled on one end only, the lifting apparatus on the two ends being actuated from one end. Kulka uses high-speed shafts with gear transmission for transmitting the power.

The latest design by Louis Eilers, Hanover, presents a particularly practical ice trap. It is operated in the most satisfactory manner by using a connecting rod to lay back the trap and another rod to raise the segment completely. Fig. 286 shows such a segment in the fabricating plant; Fig. 287, a completed weir with balance beams, the counterpoises being made of concrete. (Also see Fig. 285.)

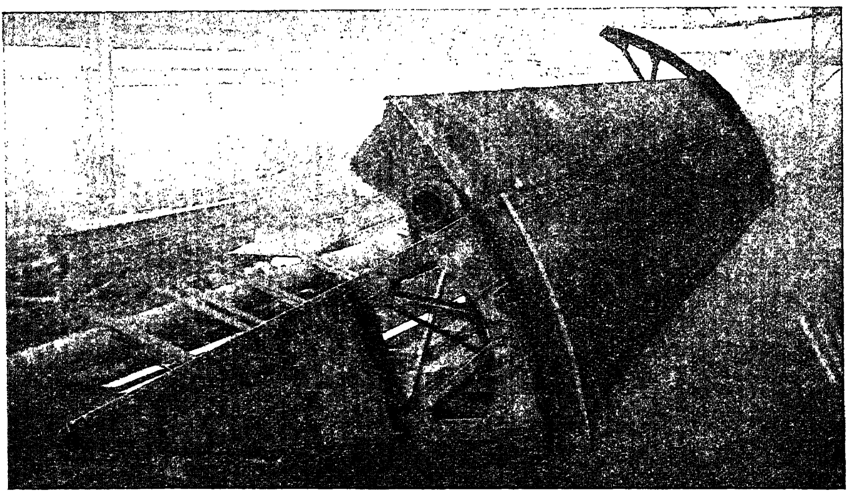


Fig. 286. Segment weir with ice trap for Boberroehrsdorf in process of construction. The far end shows the operating arm

Eilers has found it possible to close openings up to 40 m. (131 ft.) clear breadth by means of segment weirs having several intermediate supports (the Oder weir at Barteln near Breslau). Segment weirs are

[1] Dr. Kulka, *Segmentwehre.* Wilhelm Engelmann, Leipzig.

at present in close competition with cylindrical roller weirs. They are constantly being improved and may be considered one of the best types of weirs now in use.

A particular form of segment weir has been developed for closing off stretches of canal. However, the design provides nothing particularly new which has not been used in the newer types of weirs. In this design, moving the bearing further backward in a particular groove results in the lengthening of the entire structure, and therefore the structure is not to be considered as a pattern type.

2. Sector Weirs

In design, the sector weir is a variation of the segment weir, but is an outgrowth of the double-trap weir. The older double-trap weir of the Chicago Drainage Canal (not illustrated here) fulfills practically the same requirements as the sector weir. For further development in Chicago, the double trap has been replaced by the locked segment.

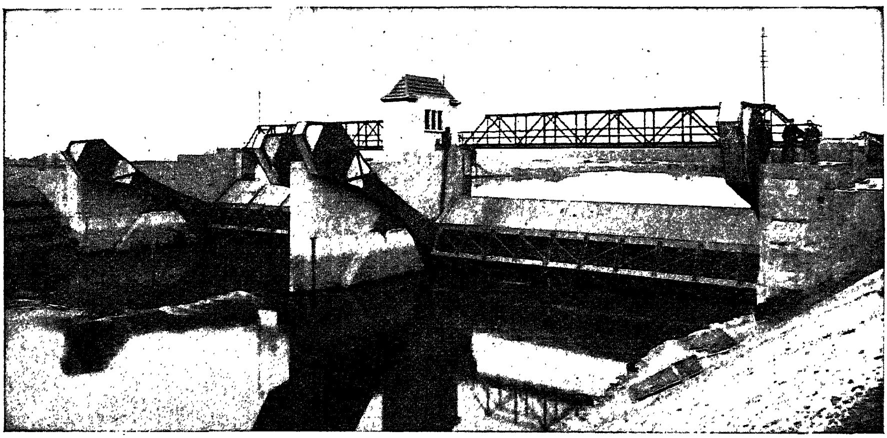


Fig. 287. Segment weir planned according to the system of Louis Eilers of Hanover. Clear breadth 22 m., damming height 6 m., actuated from one side

A larger weir of this type was constructed more recently at Hemelingen near Bremen. The weir is required to maintain the headwater level automatically at constant elevation, regardless of whether the inflow from upstream varies or the turbine plant varies its rate of water consumption, due to a change in the tidal level incurred at the tail-water side. In any case, the weir must be so constructed that it will allow variations in discharge without appreciable change in the headwater level. Figs. 289 and 290 show a weir which is supported upon a masonry

base by a hinge throughout its length. The use of a claw type of bearing (Figs. 291 a to c) was possible because the range of movement of the sector amounts to only about 45°. The closure is practically water-tight

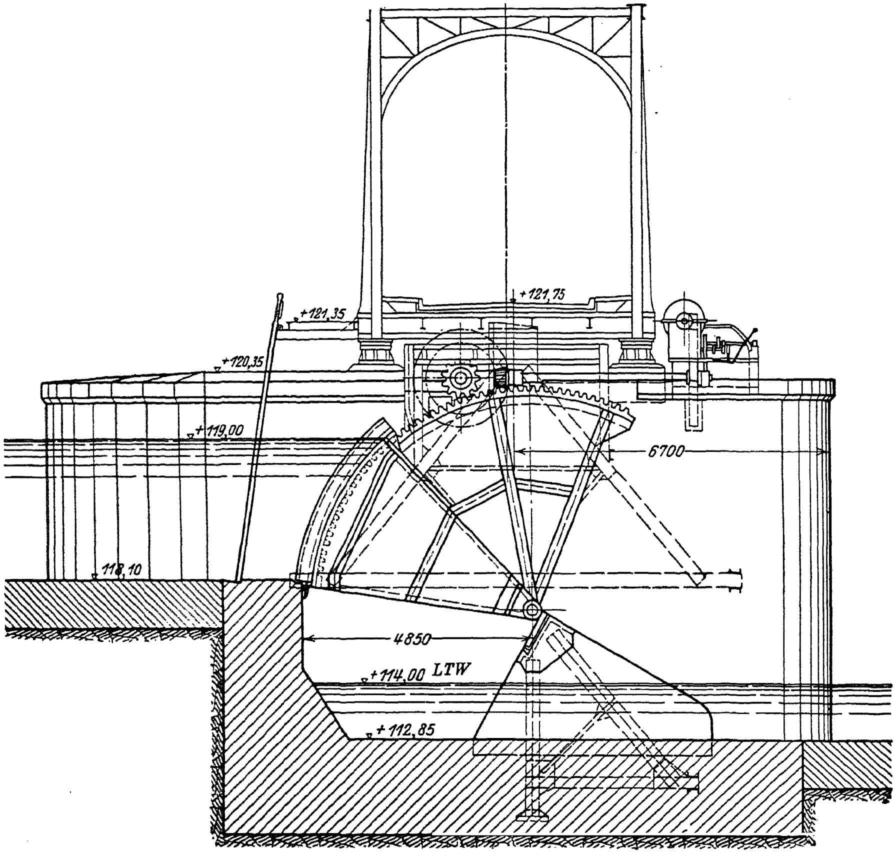


Fig. 288. Segment weir which may be lowered below the water level (Louis Eilers system)

because the shaft is continuous. The continuous bearing cheapens the steel construction. The sector is carried by the inside water pressure acting upon the downstream portion of the gate, and is maintained at the elevation required for discharging the water without changing the headwater level. The pressure against the under side of the trap is decreased by a lowering of the water stage in the chamber, thus causing the sector to sink. The pressure exerted against the cylindrical area from both the outside and inside is directed through the bearing and has no effect upon the position of the weir. Fig. 292 indicates the distribu-

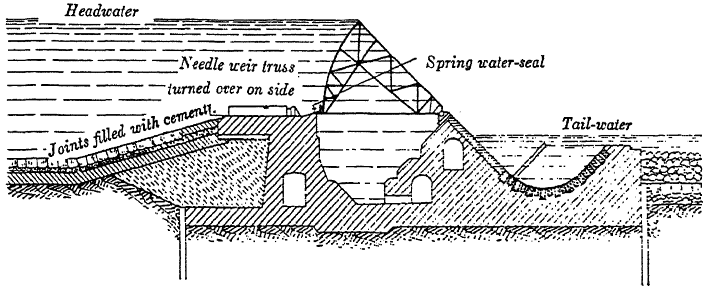


Fig. 289. Section through the weir

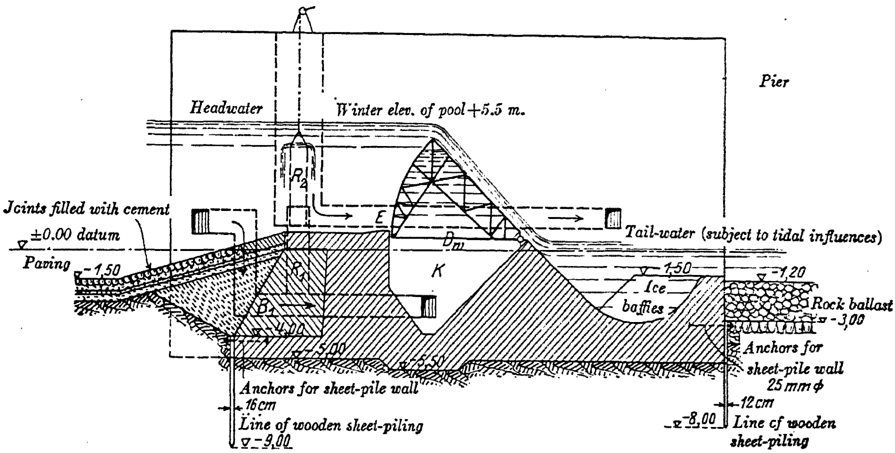
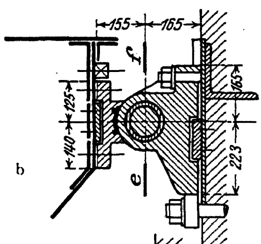


Fig. 290. Hydraulic operation of weir

Figs. 289 and 290. Sector weir at Bremen (Hemelingen)



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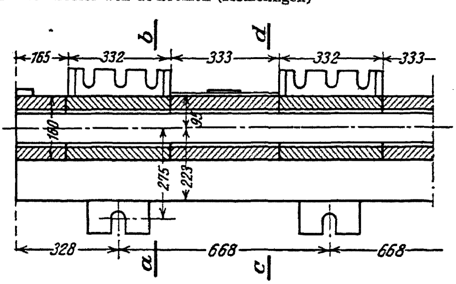

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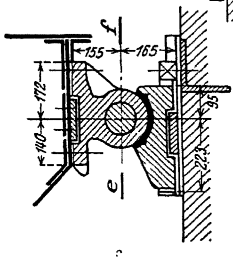


c

Figs. 291 a to c. Longitudinal arrangement of the sector weir at Bremen

a Longitudinal section through the bearing shaft
b Cross-section through the upper bearing seat
c Cross-section through the lower bearing seat

tion of forces for various water stages. The weir is controlled by a pipe sluice (Fig. 290). The installation is as follows: A canal B_1 in the weir extends frcm the headwater to the chamber K, and a shaft R_1 which extends vertically is connected to the culvert. The top of the shaft is provided with an extension consisting of a cylindrical pipe, which may be readily raised and lowered. A culvert reaches from this shaft to the tail-water side of the weir. The cylindrical pipe, called "pipe valve," hangs from a cord which may be raised or lowered by an electric motor. The motor is controlled by means of a float in the headwater. In case the headwater rises, an electric contact is made causing the motor to lower the pipe sluice, and vice versa. Theoretically, the sluice might also be actuated directly by means of the float, though no such installation exists. A variation in water stage of only a few centimeters is necessary to cause a great difference in the height of the sector. In order to cause this change to take place rapidly, the pipe valve is raised or lowered several times as far as the gate. If it remained in this position, the sector would move much further than is necessary. Consequently, the valve must be caused to move in the reverse direction. This is brought about by means of a cord connected to the crest of the weir. The pipe valve is hung from two sheaves; one of the two is operated by the electric motor, while the other is connected to the crest of the weir, a double block and tackle being inserted. Thus, if the valve is lowered too far by the motor, the crest of the weir also sinks and causes the valve to be moved in the reverse direction. A detailed description of the operation is given by Plate the inventor.[1]

The side walls of the sector are closed by a sheet-metal wall in order to form a closed space for developing water pressures below the gate. The breast water-seal and the side seals consist of spring metal. The breast seal has been greatly improved since the time of its invention, large quantities of sand having previously penetrated the seal. In one case, this penetration was so great that the weir could not be lowered entirely because the chamber became filled with

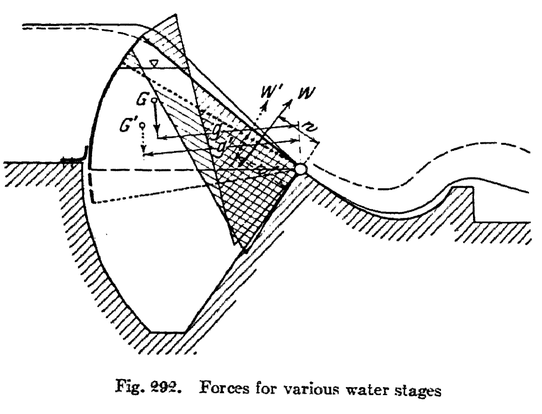


Fig. 292. Forces for various water stages

[1] Plate, *Z. V. d. I.* 1917, p. 902.

sand. The scouring equipment which had been built into the structure did not suffice at that time, and air-pressure pipes had to be used in addition so that the sand might be loosened by forcing compressed air into the chamber. The installation now operates excellently. The equipment for providing compressed air is also necessary when the weir is completely lowered and must be raised again. Under such circumstances there is no excess pressure from the headwater side. As soon as the sector is raised a few decimeters by compressed air, the headwater pressure suffices to complete the work of raising the gate. In order to hinder freezing, the side chambers may be heated electrically or by coke ovens. Special arrangements have been proposed for blowing the air from the crest of the sector. The sector is filled to the top by drawing out the air, so that the variations in the headwater may be made independent of air pressure under the gate. The entire structure is described by Koelle and Plate, German Government Engineers.[1] The weir suffered several setbacks at first but has generally proven to be an excellent type.

The sectors have a clear breadth of 54 m. (177 ft.) and maintain a maximum head of 6.5 m. (21.3 ft.). The normal head is 3 to 4 m. (10 to 13 ft.). During the summer, the head is maintained at 4.5 m. (14.7 ft.) on the gage and in the winter at +5.5 m. (+18 ft.). The tail-water may drop to an elevation 1 mm. (3.28 ft.) below the datum. The use of this type of weir, in rivers which transport a great deal of sand, is particularly to be recommended if the bed of the headwater rises 1 to 2 m. (3.28 to 6.56 ft.) higher than the tail-water bed, so that the weir chamber can be scoured out from time to time.

The fact that this weir is regulated by a headwater float actuating an electric contact indicates that similar automatic control might be arranged for segment weirs, roller weirs (cylinders), and roller-bearing sluice weirs.

In order to maintain the weir in a fixed position, a needle weir was arranged upstream and one downstream of the structure. These were necessary during the construction period and are now put in place when repairs are to be made on the sector weir. On the headwater side, the trusses are tipped and remain at the bottom of the channel; on the tail-water side, they are inserted when required. The latter are not allowed to remain in the channel in order to avoid their being damaged by ice and detritus.[2]

[1] Oberbaurat Kölle u. Baudirektor Plate, *Z. V. d. I.* 1916, p. 81.

[2] A sector weir has been constructed on the Glommen at Raanaasfoss, Norway, and another in Hammarfors, Sweden. Smaller sector weirs have been constructed as crest weirs by the Stauwerke, A. G. of Zurich, Switzerland.

3. Roller or Cylinder Weirs

The roller or cylinder weir is an invention of Dr. Carstanjen, a director of the M.A.N. (*Maschinenfabrik-Augsburg-Nürnburg*). The original roller weir was simply a hollow cylinder constructed of sheet metal with internal bracing (Fig. 293). The form shown in Fig. 293 developed considerable suction action upon being raised. The structure was therefore further developed in a manner exemplified in Fig. 295. For large heads, the cylinder would become too large and therefore too expensive. Under such circumstances, a damming shield may be placed in front

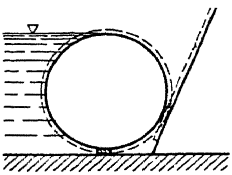


Fig. 293.

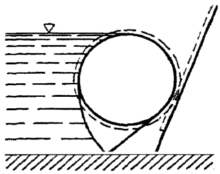


Fig. 294.

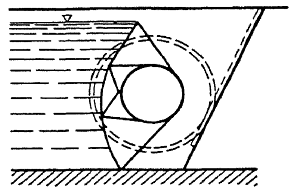


Fig. 295.

Figs. 293–295. Stages of development of cylinder weirs
Fig. 293. Pure cylindrical form. Fig. 294. Cylinder with a bill added. Fig. 295. Cylinder with damming shield

of the roller as indicated by either Fig. 294 or Fig. 295. Both ends of the roller rise upon an inclined (about 2 to 1) track which is placed in a groove of the pier. A cable on one side of the roller suffices for moving it. Both ends of the roller are constructed as gears which run on cog rails fastened to the pier. Further development of this type of weir resulted in one which could be lowered (Fig. 297) and one with a crest trap (Fig. 296). The arrangement for lowering the roller does away with the advantage of the roller weir in that the water-seal at the bottom must be accomplished by inserting a waterproofing strip. The seal may consist of a piece of spring metal, just as in the case of segment weirs, which are arranged for lowering, provided the lower part of the damming shield is designed accordingly. In the newer designs of the M.A.N., the spring-metal strip is fastened directly to the roller. There is considerable advantage in being able to raise this strip out of water.

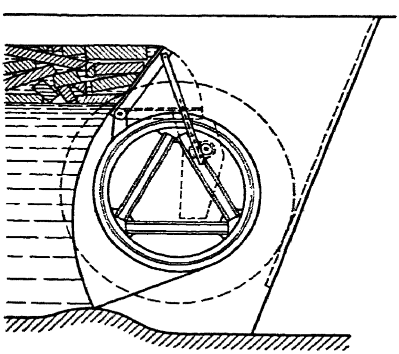
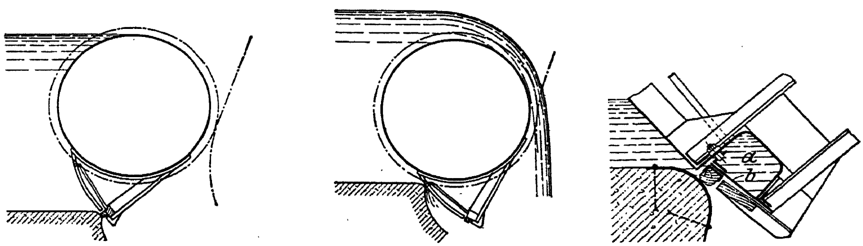


Fig. 296. Cylinder with damming shield and ice trap

The stresses in the roller are generally greatest just after upward movement has been started and before much lowering of the water level has taken place. At this time, high flexural stresses occur in the structure simultaneously with very high

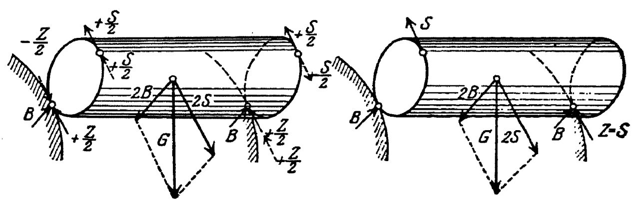
tortional stresses. The stresses developed in the roller as a result of being actuated by a cable on one side only can be shown by first considering a roller propelled on both sides. Figs. 298 a and b show a roller operated from both sides when in a raised position. The weight G, which is considered to be concentrated at the center of gravity, the latter being in the center of the roller, is resolved into two components respectively in directions normal and tangential to the track. For simplification, the operating cable is considered to act parallel to the tangent of the track. If the forces normal to the track, $2B$, are taken as a portion of G, those tangential as $2S$, the bearing pressures normal to the track on each side of the weir are B. Both sides of the roller are held tangential to the track by the pressure on the cog of the rack, the latter stress being indicated by $+.5\ Z$ and the tension in the cable by $+.5\ S$.

If it is now desired to consider a roller actuated from one end, one of the cables must be removed; this is done mechanically by applying a counterforce $-.5\ S$. This procedure results in a downward movement of the roller which, however, is resisted by the remaining bearing forces, thereby causing tortion in the roller. Equilibrium must be restored by



Figs. 297 a to c. Cylinder which may be lowered below water level (M.A.N. design).
a Normal position b Lowered position c Water-seal at the bottom

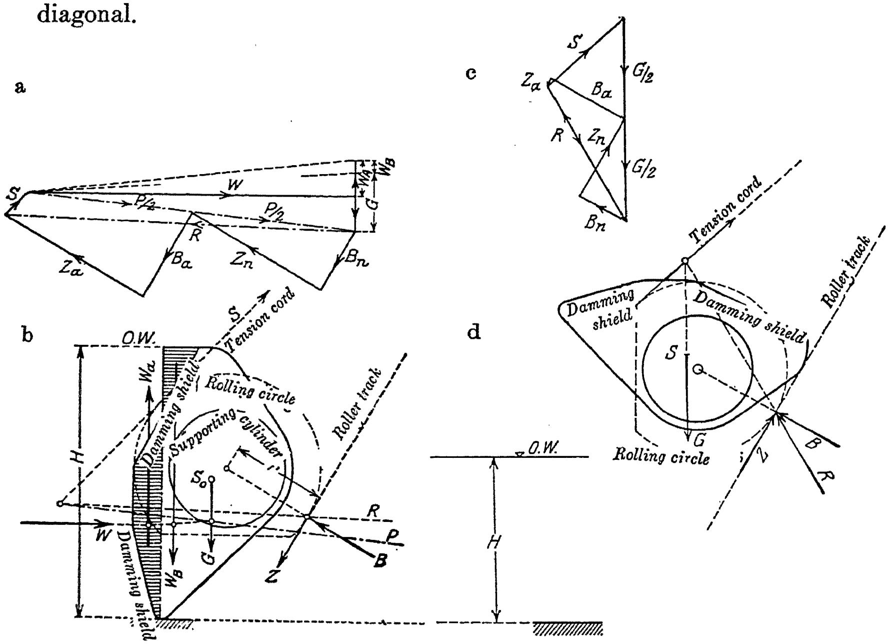
the addition of other forces; the sum of the tangential forces in space as well as the sum of the moments must remain the same as previously.



Figs. 298 a and b. Resolution of forces for conversion from two-sided to one-sided actuation

By first considering the vertical plane, it will be noted that the requirement of equilibrium of tangential forces requires an increase in the pres-

sure upon the cog equal to $+.5\,Z$ in which $.5\,S=.5\,Z$. A moment thereby arises tending to twist the roller. A countermoment must develop on the other side if equilibrium is to be maintained. Therefore, on the left side at the top of the roller, a force $+.5\,S$, and at the bottom a force $-.5\,Z$ must be applied in order to develop the countermoments. Conditions of equilibrium are fulfilled simultaneously for all other planes; at the top of the roller, the forces $-.5\,S$ and $+.5\,S$ neutralize each other; at the bottom the forces $+.5\,S$ and $-.5\,Z$ do the same. This is also true of the forces $+.5\,S$ and $-.5\,Z$ in the left vertical plane. Thus equilibrium is maintained in space inasmuch as there is equilibrium in all planes. The final forces are those indicated in Fig. 298 b, namely, the force $+S$ above at the left end and the force $+Z=S$ at the bottom of the right end. At the bottom of the left end the force upon the cog vanishes. The roller is now supported in space along the diagonal.



Figs. 299 a to d. Resolution of the outer forces of the bearing pressures on a cylinder. At the left before raising the cylinder; at the right, after raising

The resolution of the outer forces is shown in Figs. 299 a to d. At first the roller will be considered in its closed position. The tail-water pressure is neglected; the horizontal pressure resulting from the head-water is taken as W_w, the hydrostatic uplift exerted upon the roller as W_a, the loading resulting from water pressure downward upon the dam-

ming shield as W_b, the cable tension on one side as S (in this case S is not parallel to the cog pressure Z), and the reaction normal to the track as B. The forces Z and B act upon both sides; on the side of the roller which is actuated by the cable, they are designated as Z_a and B_a; on the opposite side, as Z_n and B_n. Contrary to the previous case, the weight of the roller G acts as an eccentric force. All of the known forces, the weight, water pressure, uplift, and downward pressure of the water are resolved into a single force P. This force P is uniformly distributed over the length of the roller. The force .5 P comes into play at each of the ends. The entire force P acts upon the tension cable S inasmuch as all of the forces must be overcome by this cable. Thus, for this purpose, the forces Z_n and B_n may be considered as acting in the plane of the force S, so that the system can be resolved into components in the vertical plane through S. The point of intersection of S and P is determined and the forces Z and B so fixed that their resultant passes through this point, thereby requiring an additional force R to hold the system in equilibrium. The forces R, P, and S pass through one point and can be resolved into a force triangle. The force S is found in this manner. All subsequent analyses are made considering each end of the roller separately. On the end upon which the cable acts, the known forces .5 P and S come into action; the latter must be resolved into the forces Z_a and B_a. On the free end, the force .5 P must be resolved into the forces Z_n and B_n. A force diagram in the figure shows this resolution of forces. An analysis of the roller in a raised position is performed in a similar manner in Figs. 299 c to d. It will be observed that the pressure upon the rack bearing in the first case is directed upward. This is due to the hydrostatic pressures acting against the damming shield. In the second case, the pressure upon the rack bearing is directed downward.

In order to make it possible to lower the roller below the crest of the fixed spillway, a special water-seal was invented by the M.A.N., a section of which is shown in Fig. 297 c. The arrangement provides for a longitudinal groove in the damming shield at the location of the seal. This hollow groove is connected to the headwater by means of a pipe having its open end connected with the headwater at a somewhat higher elevation than the seal. This hollowed chamber is closed by means of a spring. As the seal moves past the edge of the fixed weir, the spring makes it possible for the seal to be forced inward a limited amount. On the other hand, when the gate is completely closed, a completely watertight joint is provided. It is much more difficult to provide a satisfactory seal for a roller which can be lowered below the fixed crest than it is for a segment weir, because in lowering the roller, movement takes place

with the water-seal against the edge of the weir, thus necessitating a complicated seal. If no particular protective measures are taken, there is danger of the water-seal of the roller becoming frozen. In consequence

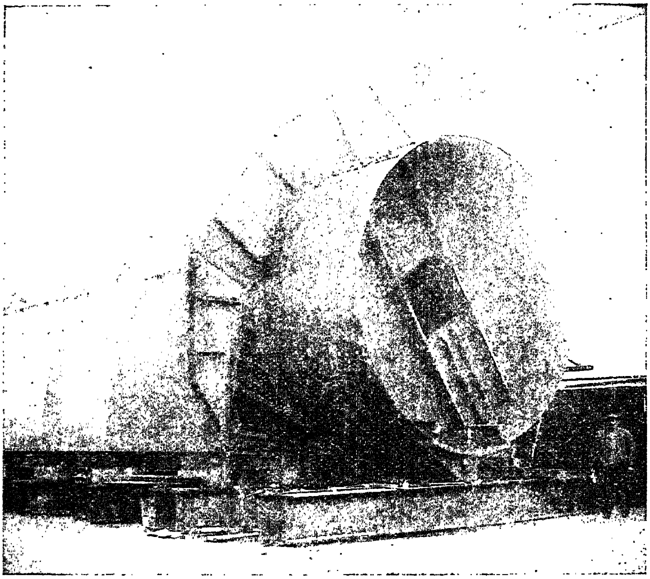


Fig. 300. Roller for the weir at Raanaasfos, Norway, in the fabricating plant. The weir has a clear breadth of 45 m. and height of 6.5 m.

thereof, the roller would become immovable or danger of breaking would be incurred. It would not be particularly difficult, however, to provide a heating arrangement in the water-seal chamber (groove), which might be kept in continual operation during times of frost, for even with such precautions certain difficulties would still remain. It may be preferable to provide the top of the roller with ice traps just as in the case of segment weirs; but even this arrangement provides difficulties in connection with roller weirs. Unquestionably these difficulties can be surmounted. In any case, at the present time roller weirs are one of the most frequently used weir types in Germany.

Roller weirs may be constructed to practically any desired breadth and height. The longest ones thus far installed are in Norway; they are 45 m. (148 ft.) long and 6.5 m. (21 ft.) high (Raanaasfos). Fig. 300 shows a roller weir in place, Fig. 302 a completed weir from the downstream side; in the latter the roller to the left is in a raised position.

A roller weir of the old type is shown in Fig. 303, a cross-section and longitudinal section of a newer type is given in Fig. 304. In the older type the supporting roller is very thick, in the newer it is much thinner;

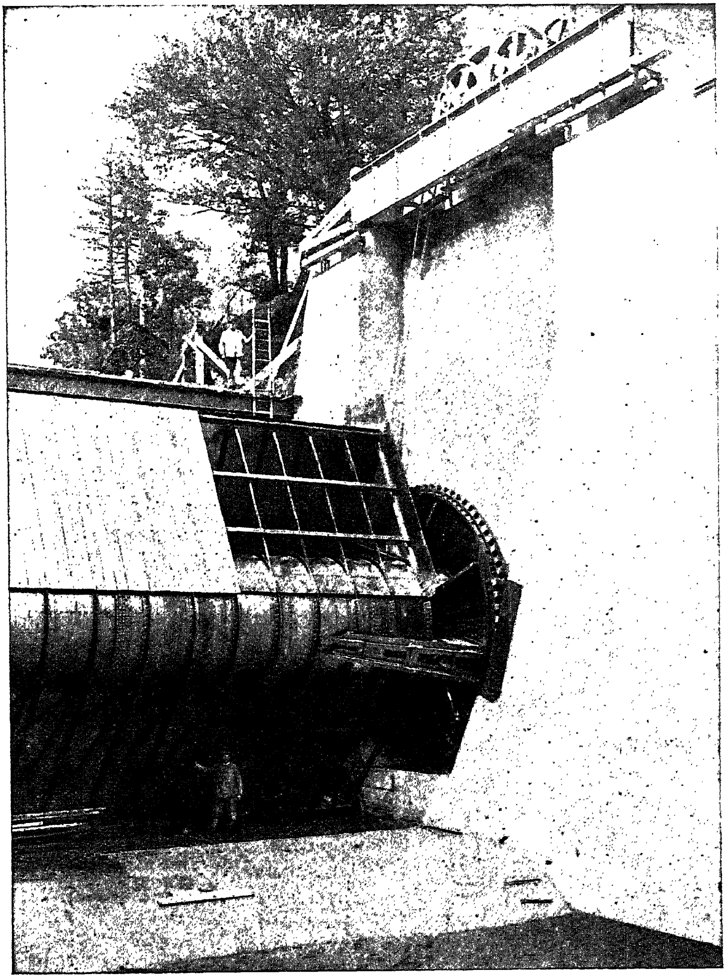


Fig. 301. Cylinder weir at Ribling. Clear breadth 13.6 m. Height 8.5 m.

the gearing is the same for both. Fig. 304 b indicates the hollowed grooves in the piers. All of these types of construction were originated by the M.A.N., who hold the patents for the roller weir.

e. Siphon Spillways and the Like

In many instances large variations in water level are considered undesirable. In case of danger incurred as a result of the water level exceeding an allowable limit, the owner of the plant subjects himself to penalty by law. In such instances, it is necessary either to provide attendants day and night or to construct automatically operated weirs. Such a weir was constructed at Hemelingen. Segment weirs of the type invented by Kulka or any of several other types of automatic high-water reliefs may be used. The siphon has certain advantages in application as an automatically operated spillway.

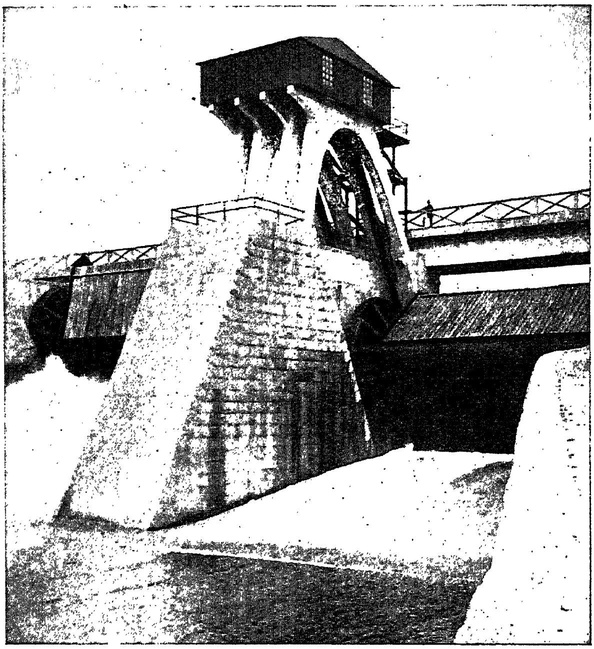


Fig. 302. Cylinder weir at Torshnofudforsen, Norway. Clear breadth

Siphon spillways have been developed particularly by a German engineer, Heyn, an Italian, Gregotti, and the Swiss firm Stauwerke A.-G. of Zurich. There is no fundamental difference in the various types of construction, so only the newer German structures of this nature will be discussed.

The purpose of using a siphon instead of the ordinary overflow spillway is to provide a head upon the crest of the weir equal to that of the water column on the discharge leg of the siphon; for example, if the effective siphon length is 3 m. (10 ft.) the velocity will be

$$v_h = \mu\sqrt{2\,gh} = 7.7\mu,$$

while for a pressure head of $h' = .2$ m., a velocity of only $v' = 2\mu'$ would be obtained. Inasmuch as μ and μ' do not differ greatly, the siphon generates a discharge velocity several times that of the ordinary overflow spillway and consequently requires a much smaller cross-section

than would be required for the usual type of weir. The siphon has proven to be an excellent means of regulation, proving satisfactory even in places where freezing weather is encountered.

Figs. 305–307 show diagrams of structures of this nature. The Heyn Company has constructed siphon units discharging as much as 150 cu. m. (5300 cu. ft.) per sec. Siphons may be constructed of steel, wood, or reinforced concrete. They go into operation automatically. When the stage exceeds a certain elevation, the water begins flowing through the siphon and carries the air contained in the siphon with it. Since both ends of the siphon are closed against further entrance of air, the chamber becomes devoid of air within a short time and discharges at full capacity. Very fine regulation is obtainable by means of siphons. Many of this type of spillway have been used in the development of the middle Isar River of Southern Germany.

f. Construction of Weirs

Weir construction is usually characterized by difficult foundation conditions. Usually the greatest difficulty consists in ridding the site of river water during the period of construction operations. Weirs are therefore usually constructed in several sections. A cofferdam is first built around a portion of the weir site and a section of the weir constructed. It is usually recommended to use one side of the cofferdam as the end of a pier of the weir. During the following summer another cofferdam may be built enclosing a section adjacent to the completed pier, after which the cofferdam surrounding the completed portion of the weir is removed. This method of construction provides no great difficulties. In case a fixed weir is to be constructed, special means must be provided for diverting the water. Inasmuch as a hydroelectric plant is usually constructed in conjunction with the weir (or a diversion canal in the case of irrigation projects), the summer water is diverted through the power plant (or diversion canal). In either case, the diversion should be constructed before the fixed weir is constructed. If this is impossible, the remainder of the construction pit for the weir must be protected by cofferdams which are much higher than for the first portion because the accumulated water must flow over the portion first completed.

A particularly difficult installation of this nature was presented in the construction of the weir on the Rhine at Rheinfelden. Here, in spite of the presence of a shell-lime substratum, the current scoured out pools (Fig. 220) to the extent that the weir had to be founded on a relatively lower substratum. The Hemelingen weir near Bremen is a particularly good example of one successfully erected (Figs. 308 a and b). Here a portion of the turbine house was first completed as far as possible,

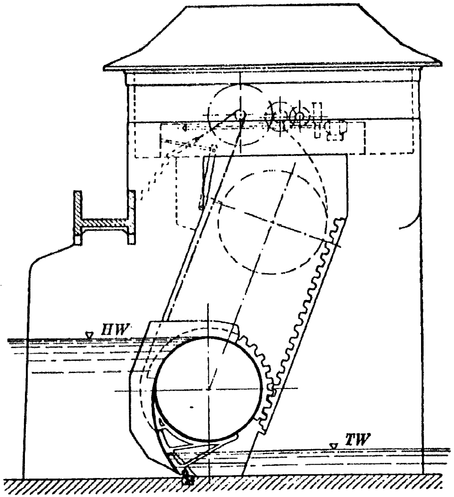


Fig. 303. Old type of cross-section with intermediate pier

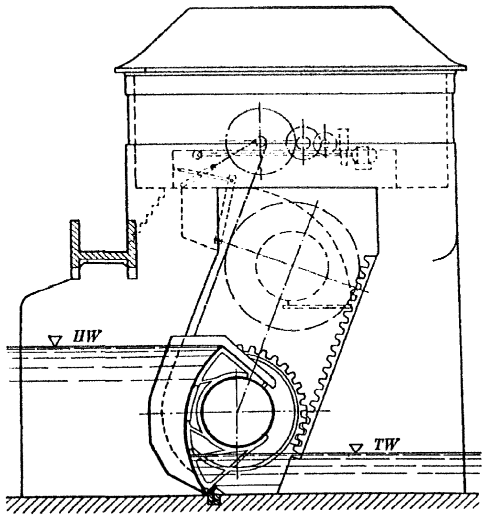


Fig. 304a. Composite type of cross-section with intermediate pier

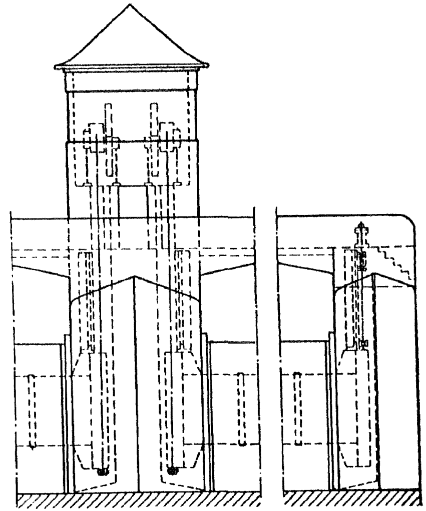


Fig. 304b. Longitudinal section through 304a

Figs. 303–304. M.A.N. cylinder weirs

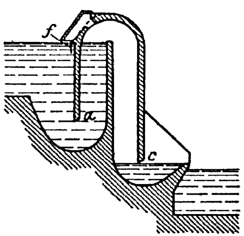


Fig. 305. Gregotti siphon

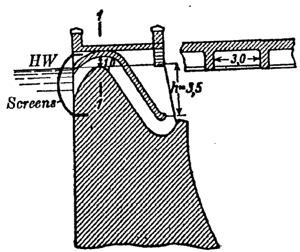


Fig. 306. Siphon by Heyn at the Edertal dam

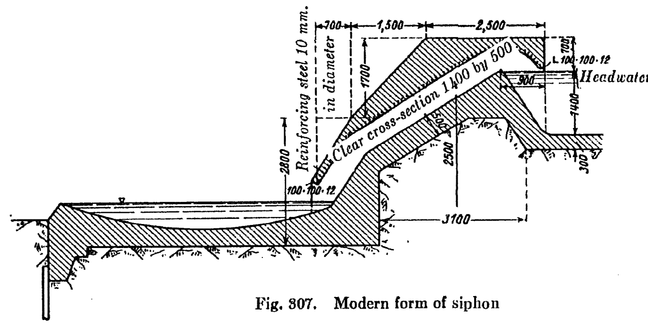
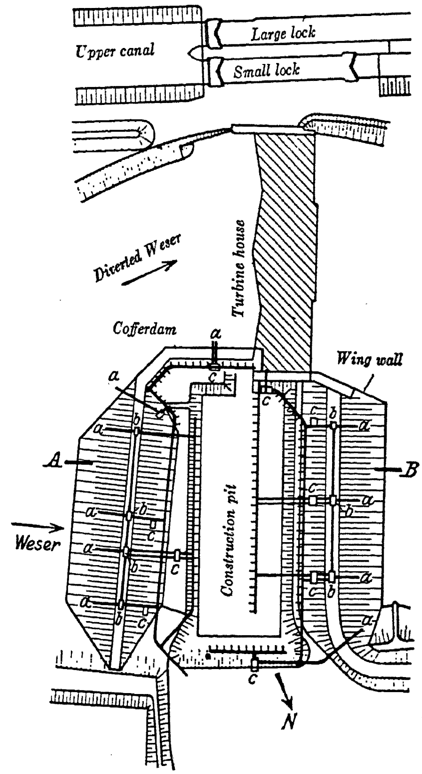
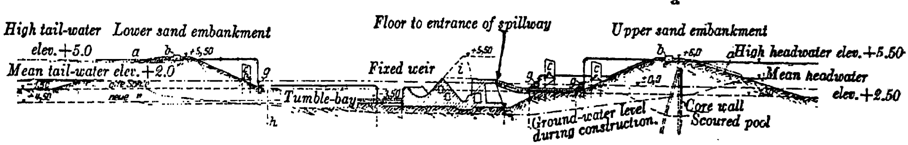


Fig. 307. Modern form of siphon

no turbines being installed, in order to provide a diversion route for the water. During the following summer the gates to the turbine house were opened and the water allowed to flow through the conduits. The building site for the weir was then closed off by sand embankments extending from the right shore to the turbine house. The upper embankment was provided with a core wall of sheet piling which was driven before constructing the embankment. Since the sill of the turbine chamber lies 1.3 m. (4.26 ft.) over the mean elevation of the river bottom, the upper dam had to be constructed correspondingly higher in order to keep out the backwater. Inasmuch as the tide rises fairly high on the downstream side (Hemelingen is now the upper end of the tidal district of the Weser), the current in the Weser was greatly reduced. During the time of high water, the gaps which had been left open had to be closed rapidly



a



b

Figs. 308 a and b. Sector weir at Bremen, showing construction pit
a Plan b Profile

by means of sheet piling. The first time this occurred, the driving of the sheet piling did not proceed rapidly enough; a gap 30 m. (98 ft.) wide was torn through the sheet piling. Upon the second attempt, however, the closure succeeded very well because the sheet piling walls were set so that they all reached above the water level. The piles could then be

lowered rapidly and rammed a short distance into the ground by three floating pile drivers. After this work had been completed, the embankment was placed. A ground-water pool was then constructed on the inner side of the dam. In this way sufficient lowering of the ground-water level was attained to provide excellent working conditions in the construction pit. The embankments were dry on the inner side; all of the water which penetrated the outside of the dam was pumped out, and did not penetrate the foot of the dam on the construction-pit side. All of the concrete work was completed in one summer and the needle-weir trusses were installed during the same period. During the following year the entire weir was completed, the needle weir serving as protection. The difficulties of construction are likely to be different for each weir; the foregoing example serves merely to illustrate how some of these difficulties might be surmounted.

g. Fish Passes and Fishways

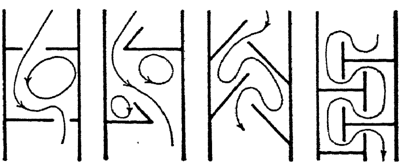
A weir constitutes an obstacle to the migration of fish if particular facilities are not provided for the fish to go over the weir. All fish, even such as are not considered migrating fish, migrate short or long stretches up and down the river according to their requirements. Migrating fish such as salmon, ocean trout, sturgeon, eel, etc., must be allowed to wander from the ocean to the rivers if they are not to be exterminated, because the rivers are either spawning places for them or at least the place where they spend the first part of their life. (The eel, for example, goes from the ocean to the river during its early life and then returns to the ocean after becoming an adult.)

All arrangements fostering migration of fish must take into account the customs of the fish. The first requirement for a fish pass is that it be located in such a place that the fish can find it. Regardless of what obstacles may prevail, fish travel upstream toward the direction from which the water flows; thus, if the pass is placed at a point at which there is no current, the fish will swim past the pass without using it. If there is a power plant directly in the vicinity of the weir, the pass should lie near the power plant because this is the location of the current which attracts the fish. If the plant lies further downstream, enough water must flow directly over the fish pass (at least at times) so that the fish will notice the current. Then within a few hours the entire assembled group of fish can wander through the pass, thereby making it necessary to allow but little water to flow through the fishway at other times. Fish frequently spend considerable time in the pass. Passes which are provided with loopholes or steps have their discharge quantity fixed. Less than this quantity of water may not flow through the fishway, but it is

possible to allow an excess of water to flow through it. A further requirement for fish passes is that all surfaces with which the fish may come in contact be as smooth as possible, so that the fish will not become injured. A fish nearly always dies if its skin is injured.[1] All ribs or transverse baffles should have rounded edges.

Arrangements causing noise or transmitting impact to the water are avoided by fish. They wander through ship locks, though mechanical appliances arranged for conveying fish have thus far proven unsuccessful. For example, the fish lock at Recken, despite its good form and apparent suitability, is not recommended because to the present time it has not attracted the fish. No fish go through the fish lock at Hemelingen, although its entrance adjoins that of a much-used fish stair.

The simplest arrangements for allowing fish to surmount weirs consist of wooden or rock fish-channels constructed with transverse walls (Figs. 309 to 312) so that the water forms numerous eddies in passing. The purpose of the transverse baffles is to lengthen the path of the water and to develop eddies so as to decrease the velocity. The depth of water in such channels should be at least 30 to 40 cm. (12 to 16 in.), the breadth about 80 cm. (31.5 in.), and the slope not more than about 1:4. Such fish-channels used to be constructed quite frequently.



Figs. 309 to 312. Simple fish passes

A similar type of structure has recently been used in connection with many weirs; for example, in Belgium, wooden or steel channels provided with special resistances to flow were hung to the movable weir. The best arrangement of this type is the Denil channel (Figs. 313 a to c), which is constructed in a hollow gap of the weir. Denil constructed wooden ribs in a channel, in a position transverse to the floor and side walls of the channel, by reconstructing an old installation previously made according to the Camere system. In the old design for a slope of 1:4 and difference in head of 3.5 m. (11.5 ft.), a velocity of approximately 6 m. (20 ft.) per sec. was allowed, a velocity which is much too high for most fish. The velocity was reduced to half of this by inserting the ribs, the breadth being reduced from .9 m. (3 ft.) to .6 m. (2 ft.). Within a period of 20 hours, nine salmon were observed wandering up the apparatus, a performance which may be considered very good for a simple layout of this type. It has been found that the velocity varies only slightly when the inclination is increased. Denil contends that a slope of 45° is permissible, but the inclination should not be made steeper than 1:3 if possible. The

[1] The skin is vital to the fish because it drinks through it.

entire layout is so simple that it is recommended for use even in connection with weirs possessing a larger fish pass. Here it should be constructed on the side of the weir away from the main pass.

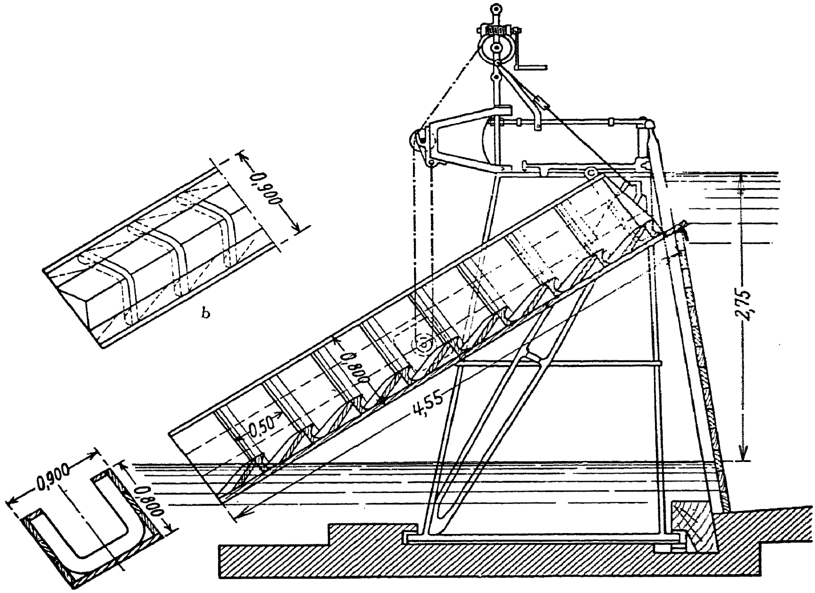
The best structures for providing fish migration are fish stairs and "wild passes." Figs. 314 a to c show the Hemelingen fish stair, which has proven to be one of the most successful of recent times. Loopholes are provided in individual basins, below each of which there is a baffle in the following channel. The baffle turns the water diagonally so as to form eddies in each chamber. Measured along the longitudinal axis, the chambers are 4.8 m. (15.8 ft.) long and 4 m. (13.1 ft.) wide, thus being basins of considerable size. The fall varies with the height of the backwater; in the winter it averages about 30 cm. (1 ft.), in the summer somewhat over 20 cm. (.65 ft.). The slope of the bottom is about 1 to 20.

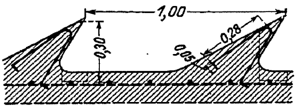
In general the following dimensions may be used: length of basin 2 to 4 m. (6.5 to 13 ft.), breadth 2 to 4 m. (6.5 to 13 ft.), water depth .5 to .7 m. (1.6 to 2.3 ft.). The fall from step to step may vary between .25 and .4 m. (.82 ft. and 1.3 ft.). The loopholes measure about .4 by .4 m. (1.3 by 1.3 ft.) to .8 by .75 m. (2.6 to 2.4 ft.). The openings at Bremen are .75 by .75 m. (2.45 by 2.45 ft.). The dimensions of the Hemelingen stairs were very generous, but the success of the structure verifies the correctness of such a design.

A very much simpler fish pass is shown by Fig. 315 a to c, which indicates the layout of the pass at the Weser weir at Hameln. In this case the chambers are very much smaller; the surface areas are each only about half that of the Hemelingen weir. The difference in elevation of the basins averages about .33 m. (1.1 ft.) for a backwater height averaging only about half that at Bremen. The pass has proven very satisfactory, showing that a satisfactory structure is possible with less elaborate design.

Fish stairs must be provided with sluiceways at the upper end, so that flow may be stopped during high-water in order to prevent an excessive quantity of detritus from being washed down the pass. The water stages in individual basins are developed automatically. If all openings are of equal size, the amount of water flowing depends upon the pressure head. Should the pressure head at the top become too large at first, more water would flow through the upper openings than the lower ones, causing the latter to fill to a greater depth and thereby reducing the head on the upper openings until finally the depth of water would be the same in each basin. Similarly, the water stages in the basins are regulated automatically with the variation in elevation in the head and tail-water.

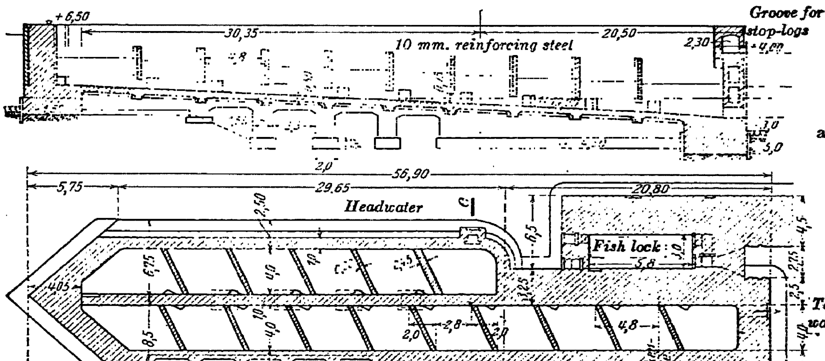
Older installations have been provided with transverse walls with-

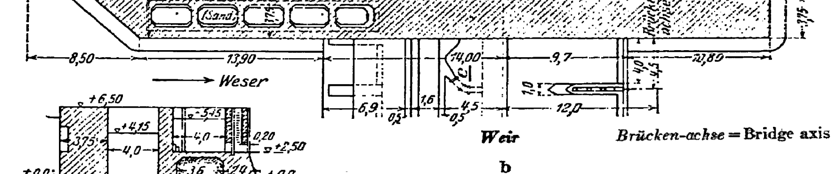




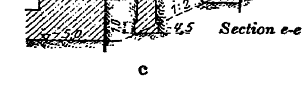
Figs. 313 a to c. Fishway by Denil

a Vertical section and cross-section b Plan of the ribs
c Section through the spurs





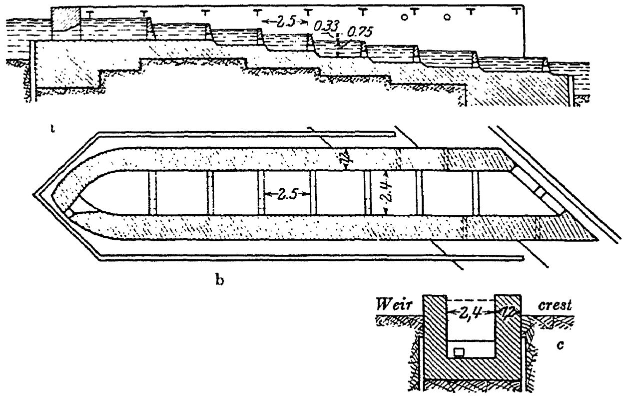
Brücken-achse = Bridge axis



Figs. 314 a to c. Fish-pass layout at Bremen (Hemelingen)

a Longitudinal section
b Plan (all loopholes lie in the center wall)
c Cross-section

out openings and a small notch cut into the top of each wall. The water passed from one basin to the other by flowing over the wall; the fish were required to leap from one basin to another. To be sure large fish can leap several meters, but stairs with loopholes are to be preferred.



Figs. 315 a to c. Fish pass in the Weser weir at Hameln
a Longitudinal section b Plan c Cross-section

The water consumption can be computed from the pressure head and size of the loophole. For large stairs, the water consumption usually amounts to .4 to .5 cu. m. (14.16 to 17.70 cu. ft.) per sec.; for small layouts, the consumption may be as low as 10 liters (.35 cu. ft.) per sec. In some instances, in order to form better attraction for the fish, a special pipe is arranged to pass water from the headwater to the tail-water in the vicinity of the lower end of the pass, thus supplementing the water coming through the pass. This arrangement increases the consumption of water; the pipe should therefore be provided with a valve so that it can be closed during times when the fish do not migrate. A water consumption of .5 cu. m. (17.7 cu. ft.) per sec. incurs a noticeable energy loss; for example, for a head of 4 m. (13 ft.) this would amount to 20 HP.

Wild-pass arrangements in Bremen were constructed upon the recommendation of Landmark, an expert from Norway (Fig. 316). It consists of a series of artificial basins of approximately 10 m. (33 ft.) diameter at the water level with a slope of 1: 2, the latter being fixed by concrete and a rolled layer of clinker. Seven such basins are provided and joined with each other and with the headwater and tail-water by channels. The channels are finished with Denil ribs constructed of reinforced concrete. The basins are arranged in such a manner that the current coming from above continually strikes the side slope, thus causing a circular eddy. Experiments indicated that in spite of the

small number of basins (seven basins compared to sixteen stair steps), the fish like to wander through the wild pass. The number of ribs should be determined by experiment. In case of incorrect arrangement, the water may discharge too rapidly and become too deep in some basins and too shallow in others.

Particular care must be taken to make it possible for the young eel to wander upstream and to wander downstream again when mature. The young (so-called) glass eel are about the thickness of a darning needle and 10 cm. (3.9 in.) long. They migrate upstream in the spring in large numbers, but are too weak to go through fish passes. They also stay clear of the open water course because the larger fish will devour them. They stay close to the shore. Wooden or clay channels must be constructed for them and lined with gravel. Only a few centimeters depth is required for them, but the water must flow very slowly. The channel may be laid at a slope of 1 : 8 to 1 : 10. The glass eels then crawl up the channel, apparently without great difficulty. Most layouts of this type, however, are not of sufficient size, because in the spring scores of glass eels may be observed to climb up the masonry walls of weirs in an attempt to reach the headwater. In order to allow the mature eel to go downstream over the weir, channels must be constructed in which considerable water is lost. The adult eel then crawls over the moist channel to the tail-water side. In case it goes through the turbines it is chopped into short pieces.

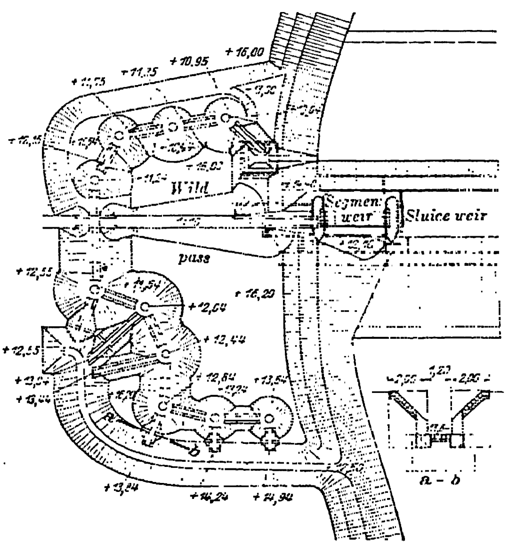


Fig. 316. Wild pass for the dam at Doerverden

The cost of fish passes is frequently very large, the stairs at Hameln costing approximately 21,000 marks ($5,000). The Hemelingen stairs, being of extraordinarily large dimensions, involved a cost of 105,000 marks ($25,000), the wild pass 62,000 marks ($14,750) (pre-war prices).[1]

[1] In view of the Prussian Weser-fishing industries, the layout was made to quite liberal proportions by the city of Bremen.

h. Comparison of Various Types of Weirs

The advantages of individual types of weirs have been enumerated at the conclusion of each description. It may be emphasized at this point that modern weirs which satisfy all requirements include only the following: double-trap weir, double roller-sluice weir without intermediate supports, and cylindrical weirs. The technical qualities of these do not differ fundamentally and the difference in cost should be the controlling factor in the choice between them. The Bremen-Hemelingen weir, which has a clear breadth of 54 m. (177 ft.), indicates that the length to which sector weirs can be constructed is practically unlimited. Under systematic development, the segment weirs can be made to span openings as large as is possible with roller weirs. For spans up to 40 m. (131 ft.), the segment weir will generally be cheaper than the other types. Louis Eilers of Hanover has designed segment weirs up to 44 m. (144 ft.) clear breadth. Furthermore, there is a certain amount of freedom in the selection of the form of supporting structure in the case of segment and sluice weirs. In the roller weir, the supporting structure must have a form more or less cylindrical in shape. The double-trap weir and the cylindrical weir have an additional advantage over sluice weirs, in that they can be constructed so as to be lowered below the water level, an item which may be very valuable in connection with closures for ship passages, particularly if a large clear height is required. The needle weir, which formerly controlled the field, no longer offers any advantage over the newer types. Even the frequently mentioned advantage that an unlimited breadth may be closed without intermediate piers is false, as is pointed out by Engels, since intermediate piers are necessary for safety. Every modern weir should be capable of allowing ice to be discharged without decreasing the height of the headwater by lifting the weir. This indispensable requirement should be given more attention than it has been given in the past. The requirements of the future are not to be found in the invention of as large a number of types of weirs as possible, but in the development of the good types to the highest degree of efficiency, simplicity, safety, and economy.

PART FIVE—SHIP LOCKS

A. GENERAL AND HISTORICAL

a. Definition

The origin of the ship lock practically coincides with the invention of the chamber lock. A chamber lock is a space which is sufficiently large to receive one or more ships and which is closed off from two variously elevated water surfaces by at least one gate on each end. This space makes possible the lowering or raising of the water surface between the two adjacent water planes without resulting in great changes in the outer water levels. By raising or lowering the water level in the chamber, ships can be lifted or lowered vertically from one outer water level to another. Either of the two gates can be opened when the water surfaces on both sides of the gate are approximately at the same elevation. The forerunners of chamber locks, which were built in the form of single gates in dikes along the North Sea about 1000 A.D., are to be classified as locks because they were opened at times when the water elevation on both sides was equalized. On the other hand, forerunners of this chamber lock, which consisted of single gates in solid weirs, are simply our present-day ship passages through weirs and do not deserve the name locks. They are to be classified as movable weirs.

The criterion of ship locks is that they are to provide passage for ships but not for water. When simple gates were used in locking ships along streams, the passage of water was unavoidable; but in the case of simple dike and dock gates this was not necessary. Locks are divided into these two classifications where the purpose rather than style of structure is considered. In order to operate a simple automatic lock gate (dock gate), it is necessary that the temporarily lower outside water level rise above the harbor water level. This condition occurs regularly in the case of North Sea ports which are provided with entrance gates. The gate remains open during the time of high outside water elevation. It resumes its purpose after the outer water level has fallen below the normal harbor water level. Accordingly, a single lock gate may connect a harbor basin with the sea during periods when the tide is up, but the gate cannot render the harbor accessible at all times.

b. Historical

Investigations of Dr. Wreden, Hanover, indicate the probability that the chamber lock was invented almost simultaneously in Germany,

Holland, and Italy. There is some possibility that Italy was influenced by the northern countries, but this has not been proven.

Wreden gives the following information in his publication on the forerunners and origin of the chamber lock.[1]

The oldest construction dates or reports concerning ship chamber locks are:

1. In Holland, perhaps the year 1203, probably however 1413. A decision concerning the meaning of the words *verlaat* and *kolk of schutthinge* at that time is a task in philology (Fig. 317).

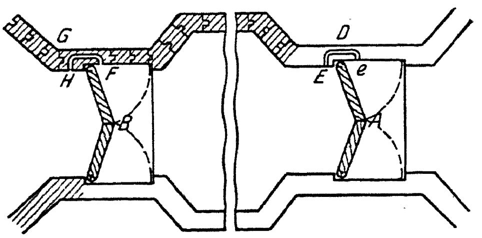
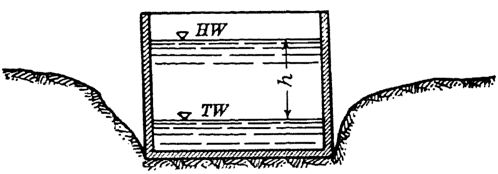


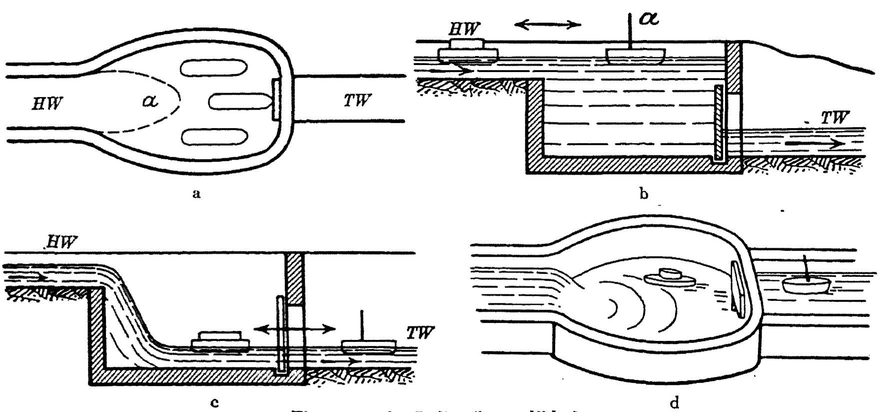
Fig. 317. Oldest form of chamber lock in Holland (according to Stevin); constructed long before 1600



Figs. 318. Coffer lock on the Stecknitz River (Germany)

2. In Germany, the year 1325 marks the time of the first construction of double damming locks, 1398 or 1448 as the year of construction of the first chamber lock (Fig. 318). The first solid (stone) basin-shaped chamber lock was built in 1569.

3. In Italy, a shell-shaped lock (conca) was built in 1420, in 1452 a double damming lock (proposed by Leone Battista Alberti), and in 1497 the chamber lock closed at both ends was first built with contrivance for filling and emptying it. Leonardo da Vinci was first to design and construct such a lock on a technical basis (Figs. 319 a to d).



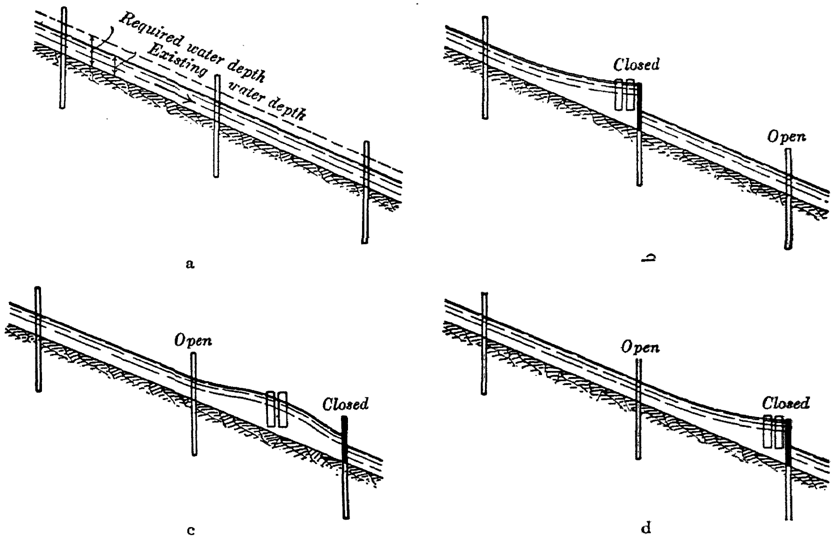
Figs. 319 a–d. Italian "mussel" lock

a Plan view of lock (x overflow area) b Chamber filled c Chamber empty d "Conca in Stra" at Padua 1481 (according to an old etching)

[1] Wreden, *Vorläufer und Entstehen der Kammerschleuse usw.*, 1919, Julius Springer, Berlin.

It became imperative to develop the lock at that time because water courses which were formerly continuous were being interrupted by dams (dikes along the sea or fixed weirs in streams) and because it was desired to cut through land separations between neighboring variously elevated water courses for the passage of ships. In its oldest form, navigation resorted to the system of reloading. The freight was brought from one water level to another instead of the ships. The next step forward was the invention of ship crossings consisting of inclined planes, for the very small vessels of that time (up to 20 tons capacity), the movement of ships over the inclines being facilitated by rolling them or by placing a layer of clay on the slope.

The famous canal planned by Charlemagne, in about the year 800, between the Rhine and the Danube (Rhine–Main–Regnitz–Rednitz–Rezat–Fossa Carolina[1]–Altmuehl–Danube) could scarcely have been planned without such oblique planes if reloading the wares from one water elevation to another was to be avoided. An improvement in the

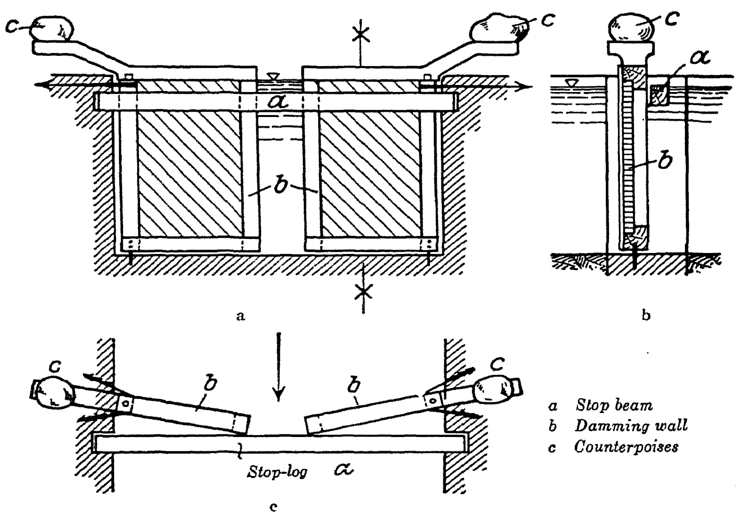


Figs. 320 a–d. Transportation on waves (Stecknitz route)
a Condition without backwater b Weir 1 closed c Weir 2 closed. Ships travel on the crest of the wave from 1 toward 2 d Weir 2 closed. Ships await opening of weir before continuing journey

procedure was rendered possible in rivers by constructing dams for raising the water level. Thus the passage on waves was invented (Figs. 320 and 321). In this process the ships were assembled above the weir, then a gate in the weir was opened allowing a rather large quantity of water

[1] The Charlemagne Canal is located near the Grönhart Station, a small station 4 km. (25 mi.) from Treuchtlingen in Bavaria.

to flow off until the waterfall was somewhat flattened out. The ships were then permitted to move down the stream on the wave. Similarly, by releasing additional waves it was possible to further the ships to the next weir and so on. In the vicinity of particularly unfavorable crossings, it was necessary to construct two weirs close together. In bringing such weirs very closely together a short pool was formed serving the same purpose as the lock chamber. A passage of this nature was constructed in the Stecknitz route which connected Lübeck with the Elbe, forming an important outlet for Lübeck's salt trade. The upper lock on the Stecknitz is mentioned in a document by Duke Albrecht IV of Saxony, 1336; however, the Stecknitz route antedates this considerably. On the Stecknitz, a chamber lock was devised, the so-called coffer lock; the latter doubtless was an outgrowth of experience with locking ships through the two closely connected sluice gates. Fig. 318 illustrates the type of lock used on the Stecknitz.



Figs. 321 a–c. Damming locks in the Stecknitz route
a Front elevation of gate b Section c Plan

The chamber lock was developed in an entirely different manner in Holland. According to Stevin, Fig. 317 shows the first chamber lock in Holland. Although it existed long before 1600, it contains two pairs of mitering gates, A and B, and by-pass culverts, C, D, E, and F, G, H, which were closed by small gates. The forerunner of Italy's chamber lock is the "shell lock," in which a natural waterfall in a stream was provided with a movable lock (the lower sluice gate) (Figs. 319 a to d).

By adding an upper gate, a chamber lock was formed. This was invented in Italy by Leonardo da Vinci and described in his writings (1497). Chamber locks were being used in Germany and Holland at the time Leonardo designed the lock. His most important practical invention relative to locks was a contrivance for filling and emptying locks by inserting gates, a method which was formerly unknown. This invention was the most important step in the development of the lock and was instrumental in making the chamber lock a practical and useful device.

B. CLASSIFICATION OF LOCKS

a. Simple Locks

As shown by historical development, chamber locks are the outgrowth of simple gate locks (lock gates). The lock gate is to the present day the simplest form of lock used. This type is employed for dike gates and harbor gates along seashores subject to tides, etc. Fig. 322 shows a simple harbor lock, the so-called dock lock, which to this day is in many cases the sole connection between England's harbor basins and the sea. The harbor is accessible only when the outer water level has risen above the harbor water level. The lock consists of a masonry or concrete

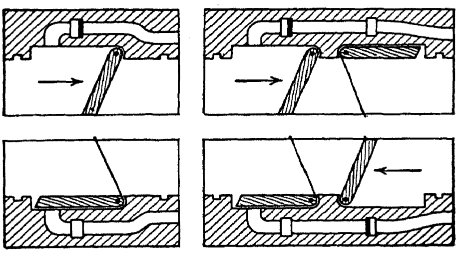


Fig. 322. Fig. 323a.

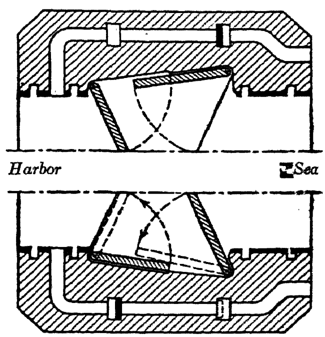


Fig. 323b

Figs. 322–323. Gate arrangement
Fig. 322. Single mitering gate, in closed and in open position
Fig. 323 a. Double mitering gate (safety gate)
Fig. 323 b. Double mitering gate (gates overlap)

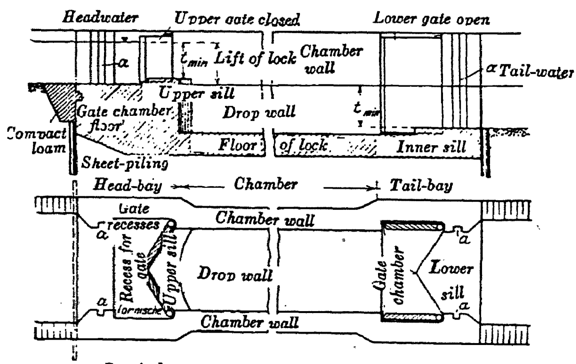
passageway which is protected against undermining by sheet piling. The frame of the passage is constructed with great care since it forms part of a dike which protects both the harbor land and lowlands from the floods of the sea. In this passage are installed one or several pairs of mitering gates which swing into recesses in the wall when open (Fig. 323). The outer gates are used during tidal floods when the sea water is to be kept from the harbor basin. During normal tides up to the customary spring tides, the water is permitted to flow into the harbor. A pair of gates may be replaced by a floating or rolling pontoon. A somewhat

different form of lock results by interchanging two pairs of gates (Fig. 323 b). This arrangement brings about a saving in masonry work since only one recess is required instead of two. However, the depth of the recess and also the length of the gate folded back are greater than in the arrangement in which the two pairs of gates do not come into contact with each other. The arrangement to be preferred is dependent upon the cost of construction. By-passes are often dispensable in these gates.

b. Chamber Locks

The foregoing describes individual parts rather than complete locks, which are called chamber locks. The latter may be found in a great variety of forms; namely, as a single lock, double lock, twin lock, barge-train lock, etc.

A simple chamber lock of modern form is shown in Figs. 324 a and b. The terminology varies according to whether they are locks in rivers and canals, or whether those in the sea are considered. A lock consists of the head-bay, tail-bay, and the chamber. The customary terminology is indicated in the figure, thus making repetition unnecessary. The port sill is the floor stop against which the gate rests.



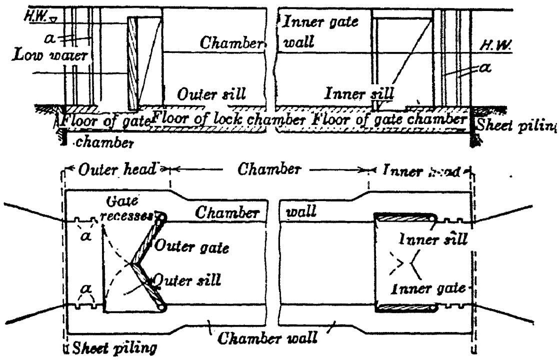
Fornische=gate recesses

Figs. 324 a and b. One-way river lock

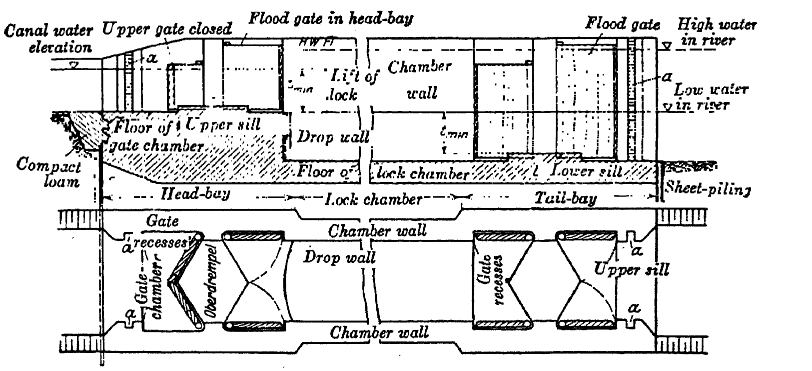
There is an important difference between river or canal locks and ocean locks, as the head-bay of river and canal locks usually lies considerably higher than the chamber. The transition from the head-bay to the chamber requires a curtain wall, which was made arch-shaped before the age of concrete, but is now usually constructed as a straight reinforced concrete wall. The highest point in the floor of the head-bay and bed of the tail-bay, respectively, is the port sill for the lock gate in each case. The upper edges of the sill must lie at least as low as the bottom of the river, canal, or harbor. The sill is frequently laid considerably deeper in order to allow for the possibility of deepening the adjacent water courses.[1] In front of the gate chambers, the bed of which is lower

[1] Because of regulation, the low water level of the Weser River has fallen 1 cm. (.394 in.) annually during the last few decades. In the Oder River the LW below Breslau has fallen 5 cm. (2 in.) annually so that finally the lower port sill lay exposed at LW.

than the rest of the floor of the lock, lie the outer locks. They should each be provided with an arrangement for a needle weir, a bulkhead weir, or a pontoon, so that the bay can be closed temporarily for the purpose of making repairs. These closures must extend above the highest water level encountered during the time repairs are made. Thus, for example, a needle weir in the tail-bay needs only to extend to the tail-water level because this is the highest water level arising at this point during repairs. Although mitering gates have been practically abandoned, a great number of them are still being built.



Figs. 325 a and b. One-way sea lock



Figs. 326 a and b. Descending lock to river, the lock having flood gates, gates opening in both directions

The important difference between canal locks and sea locks is made evident by Figs. 325 a and b. The outer bay and inner bay of the sea lock correspond respectively to the head-bay and tail-bay of the canal or river lock. If a chamber lock is to be capable of maintaining the higher water level at either end, it must be provided with at least two pairs of gates in each direction (Figs. 326 a and b). Such two-sided locks may become necessary if a canal empties into a river where the water level of the canal lies above the low-water and below the high-water stages of the river.

c. Double Locks and Train Locks

The fundamental types of layout for locks are presented on the following page. The simple locks are represented by Figs. 327 and 328. Formerly double locks were employed (Figs. 329 and 330) where it was desired to lock a number of ships at the same time. In these locks the entrance and exit are usually displaced with respect to each other and

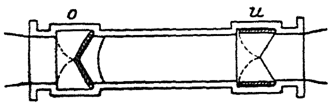


Fig. 327. One-way lock (gates opening in one direction)

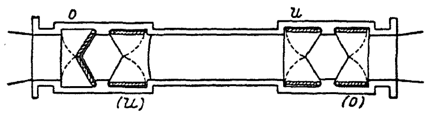


Fig. 328. Two-way chamber lock

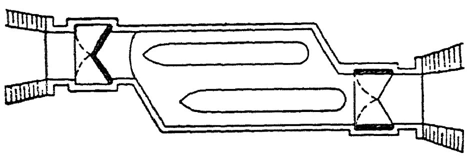


Fig. 329. Displaced lock (double lock)

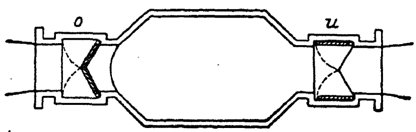


Fig. 330. Basin lock for three ships

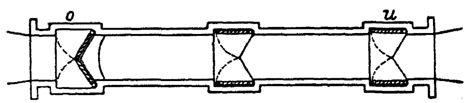


Fig. 331. Train lock with center gate

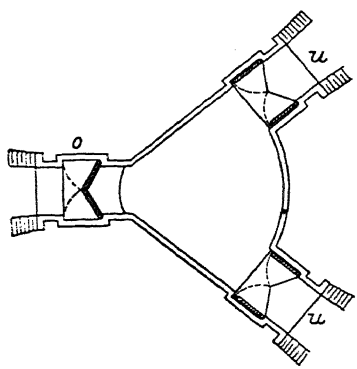


Fig. 332. Switch lock

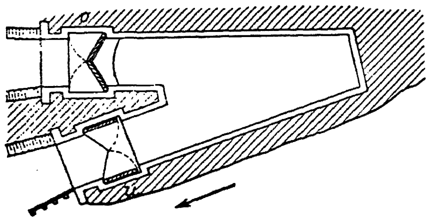
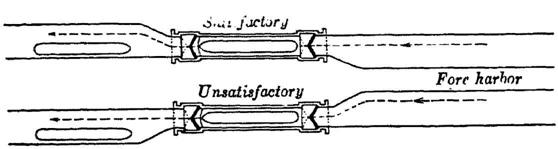


Fig. 333. Pocket and head lock



Figs. 334 a and b

the lock proper. Multiple locks of this nature are now seldom constructed, but along the sea they are suitable under certain circumstances without displacement of the ends. They facilitate traffic of large and small ships at the same time. The large ship enters first and leaves first. The

Kaiser lock at Bremerhafen is one of this sort. The train lock (Fig. 331) serves the same purpose as the double lock and is far better for inland navigation. It permits locking whole barge trains in one operation,

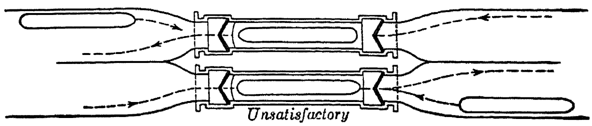


Fig. 335a

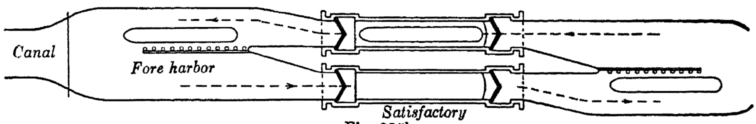
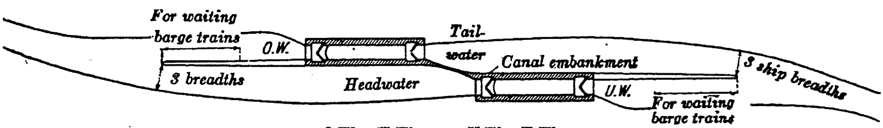


Fig. 335b

Figs. 334 and 335. Suitable and unsuitable arrangement of entrance and outer harbors for single and twin locks



O.W.=H.W. U.W.=T.W.

Fig. 336. Separating arrangement of locks to be used in mining districts and the like

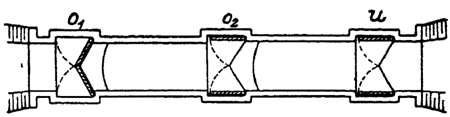


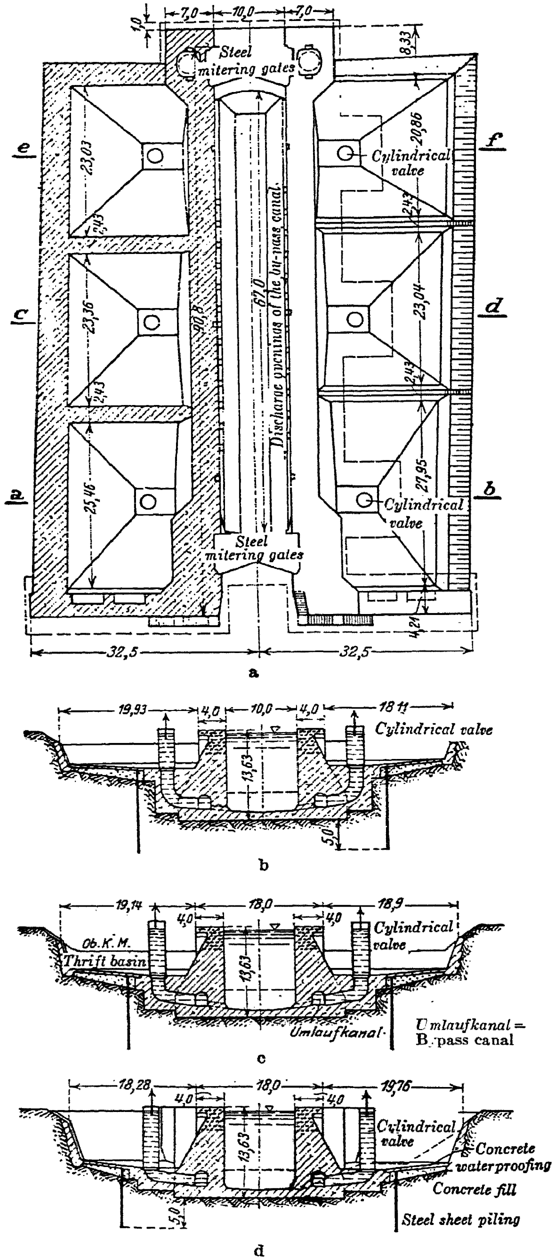
Fig. 337. Lock flight

whereas locks of the type shown in Figs. 329 and 330 necessitate the breaking up of a train during the passage. The train lock for two ships is more expensive than the double lock for two ships, but has the same water consumption and saves considerable time in locking. Commercially and scientifically it is far superior to the double lock and is now universally constructed for heavy traffic. Large train locks attain lengths of several hundred meters; thus, for example, the Bremen train lock in the Weser River at Hemelingen has a clear length of 350 m. (1148 ft.) and a clear width of 12.5 m. (39.5 ft.).

The basin lock (Fig. 330) is a type which allows the locking of a number of ships simultaneously. In this layout the usual narrow chamber is replaced by a basin-shaped space (similar to that of the shell lock, Figs. 319 a to d). This lock is of historical interest only.

d. Multiple Locks, Flights, Mechanical Lifts, etc.

Handling traffic from two directions through one lock often results in congestion and, in case of damage to the one lock, the traffic becomes entirely paralyzed. Nowadays, therefore, wherever financially possible, double lock systems are erected. The usual type of layout on good subsoil is the twin lock (Figs. 335 a and b), which consists of two locks lying side by side and having a common middle wall. The use of a twin lock instead of two single locks results in a saving in masonry amounting to a little less than the cost of a lateral wall, and considerable saving in earthwork. Frequent reference has been made to the dangers that threaten the entire structure if a lock chamber becomes undermined or if some similar misfortune were to occur. In spite of this undeniable danger, twin locks have been built and will continue to be built in the future. The advantages of saving in building cost, supervising the operation of both locks from one point and the decrease in lock personnel are so great compared to two single locks that the inherent dangers of destruction of the twin locks is negligible.



Figs. 338 a–d. Thrift lock at Niederfinow with open thrift basins
a Plan b Cross-section e and f
c Cross-section c and d
d Cross-section a and b

In Germany twin locks have been used on many projects, except where the subsoil contains great dangers within itself, such as occur in the mining regions (Westfalen, Rhein-Herne-Kanal). Here the locks were built as single units paralleling each other at a great enough interval so that an accident in one lock would not necessarily be transmitted to the neighboring lock (Fig. 336 of the Rhein-Herne-Kanal).

The arrangement of the ground plan leads to the further classification of locks into switch locks, pocket locks, basin locks, and turn locks. The nature of these is explained by Figs. 330, 332 and 333; they are now only of historical interest.

Recent practice in canal construction favors constructing as few lifts as possible, the total lift being concentrated in a minimum number of locks. Locks with lifts of from 15 up to 36 m. (from 49 up to 118 ft.) are now being constructed. Formerly large differences in elevation were surmounted by locks arranged in flights (Fig. 337). In the flight, the lower end of the upper lock also acts as the upper end of the next one, etc. Shaft locks with reserve basins (or thrift basins) are now built in place of flights. Fig. 338 shows such a lock with three open reserve basins on each side. Attempts to conserve water then occasioned the invention of other types of water-saving locks. Among these are the displacement lock by Schnapp and Gerstenberg, and the float lock by Schneider. Both are surpassed by the by-pass lock series invented by the author and by Professor Proetel's various thrift-basin locks which theoretically function entirely without water consumption just as the Schneider lock. The Proetel lock may be applied to single chambers, whereas the Schneider lock as well as the by-pass lock presupposes a double two-step lock flight.

Great heights can also be surmounted by inclined planes and by mechanically operated lifting devices. These are discussed individually at the end of this chapter.

C. LOCATION AND DIMENSIONS OF LOCKS

a. The Location of Locks

The location of a lock for the most part is determined by the route of the waterway. Consequently, when the route is once fixed, there is not a great amount of choice in the location of the locks. Full consideration should still be given to the lock site when determining the canal route. Thus, if possible, canal locks should be built near the intersection of important highways or the intersection of the canal with the railroad. With such a layout it is frequently possible to build the railway or highway crossing over the lower or upper end of the lock with but slight

change of position. This procedure invariably results in great reduction of bridge construction expenses. Descending locks leading to rivers, if practicable, should not be placed too far in the slope if there is danger of landslides. If the consideration of railroads or main highways demands, the lock may be constructed along the slope provided the site is satisfactorily stratified. In rivers (canalization) the lock has usually been built along the convex side, where it is least endangered by floating ice but is subject to the danger of sediment movement. Inasmuch as the stream is dammed up except during floods, the possibility of the lock's being sanded up is small compared to the circumstance for the concave side when there is no obstruction to free flow of the stream. However, a certain amount of dredging becomes necessary. Nowadays locks are often built in the concave side (the German Main). Lock canals should be located so that they empty into the river at an oblique angle of about 20 degrees. Locating lock canals at very obtuse angles is poor design. This condition was unavoidable at Bremen because of the demand on the part of Prussia. If such a site is unavoidable, it appears expedient to curve the locks and eventually also the outer harbors, although hitherto such a measure has been considered inadmissible. A number of curved locks have been built above Frankfurt in connection with the canalization of the Main River. The curvature of the lock usually requires consideration only in the case of rather long train locks.

The plan of outer harbors is just as important as the lock development. Outer harbors must have a length at least equal to that of a train lock, particularly when a lock order of precedence comes into consideration, and this is almost always the case in heavy traffic. In rivers, the stream itself can often take over the function of an outer harbor,

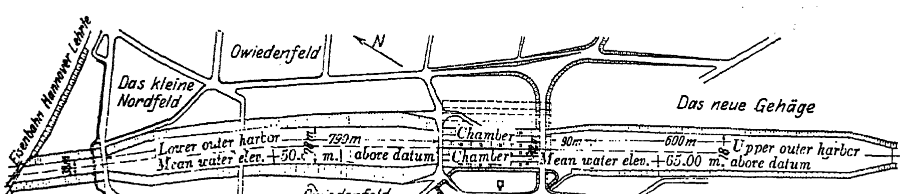


Fig. 339. Outer harbors of the lock at Anderten (near Hanover, Germany)

but here, too, it is expedient to construct good outer harbors. In canals, a satisfactory outer harbor is formed by adequate widening. As a rule, the outer harbors above and below canal locks should be made of equal length, because in the course of time an equal number of vessels must

come from both directions. (See lock arrangement at Anderten, Fig. 339.) The upper outer harbor in rivers should be longer than the lower harbor since ships from above enter with the current. Long river cut-offs provide suitable locations for long outer harbors. For the latter, in order to diminish the quantity of earthwork, the lock should be located far enough downstream to provide just the amount of space required for the lower outer harbor. The plans are greatly dependent upon the configuration of the land, and the foregoing statements should be considered only as guides in the functional design.

Sea locks should be so planned that the lock gates are not exposed directly to the highest waves. The movement of sliding pontoon gates may be rendered difficult by large waves which progress in the direction of the axis of the lock. The configuration of the land is also important in the layout. Figs. 334 and 335 indicate the proper arrangement of the shore of an outer harbor with reference to the lock.

b. Dimensions of Locks

The choice of dimensions for locks places a particularly heavy responsibility upon the designing engineer. The danger of selecting unsatisfactory dimensions for various sea-ship locks is doubtlessly least serious. Although the development of navigation after some time may be such that the lock is too small for the largest ships, its presence has not been superfluous. The dimensions of ocean vessels vary to such a degree that a lock which is too small for the largest ships still retains its importance for ships of average size. (The locks of the Manchester Sea Canal, which are discussed elsewhere in this text, provide an illustrative example.) Circumstances are less favorable for inland locks because both large and small ships are often joined into a single train. Consequently, inland shipping is handicapped by locks which are too small. Since in large canals, particularly highland canals, as many as fifty to one hundred locks[1] are required for a single route, it is obvious that the erection of a new type of lock of greater size would not be feasible very soon after completing the original project. Large canal links, such as the German Midland Canal, influence the inland navigation of whole countries, because of the great river regions connected by them, so that the lock dimensions of these canals must be given very careful consideration.

The dimensions of the modern large canal locks of Germany are as follows: width 12 m. (39.3 ft.), sill depth 3 m. (9.8 ft.), length of locks for one ship (a freight barge with towboat) 100 m. (328 ft.), for train

[1] Sixty are proposed for the Main-Danube Canal in the design of 1919 (report by Kölle).

locks [two 80 m. (262 ft.) barges with towboat] 225 m. (738 ft.). The 12 m. (39.3 ft.) locks are built to receive ships of 10.5 m. (34.5 ft.) width (over all), and either two of 80 m. (262 ft.) or three of 65 m. (213 ft.) length. The steam tugs have a length of from 15 to 23 m. (from 49 to 75 ft.). The second Rhine lock along the Rhine-Herne Canal has a 13 m. (43 ft.) width because of the large ships on the Rhine. The dimensions of large ocean locks are presented at the end of this chapter.

After the size of the ship for which the canal is to be built has been decided upon, consideration must be given to the clearance in the locks. The dimensions with which the shipbuilder is concerned differ from those with which the structural engineer deals. The shipbuilder states the measurements of the ship without washboards, the structural engineer with washboards. In the case of one important lock, the structure was built in accordance with figures of the shipbuilder, and after completion of the lock it was found that the ship was wider by the width of the washboards than had been assumed by the Board of Public Works. As a result, the largest steamer then had but 5 cm. (2 in.) clearance on either side instead of 15 cm. (6 in.). Locking caused no difficulties whatever. The clearances in Table 3 are to be recommended.

TABLE NO. 3

CLEARANCES FOR LOCKS

Type of Lock	Total Clearance*												Remarks
	For Width				For Depth				For Length				
	m.		ft.		m.		ft.		m.		ft.		
	from	to	from	to	from	to	from	to	from	to	from	to	
Lock of Inland Canals.....	0.2	1.50	0.7	4.92	0.2	1.0	0.7	3.3	3.0	10.0	89.	32.8	Abundant clearance guarantees less loss in time because it reduces the caution necessary on entering.
River Locks...	0.2	1.50	0.7	4.92	0.3	1.0	0.9	3.3	4.0	10.0	13.1	32.8	
Ocean Lock with Chamber	2.0	5.0	6.6	16.4	0.5	2.0	1.6	6.6	5.0	15.0	16.4	49.2	
Plain Dock Locks......	3.0	5.0	9.8	16.4	0.5	2.0	1.6	6.6	..	..	..	..	

*Figures indicate the sum of clearances, respectively, for two sides and two ends of locks.

An increase of the depth is very noticeable in the construction cost, but the lateral dimensions can be increased without changing the construction cost materially. Increase in length and breadth, however, frequently plays an important part in the water consumption of inland canal locks. This may increase the cost of operation but may be economical in that it reduces the loss of time. A similar condition obtains in river locks in case a water power plant forms part of the layout. Power economics suggest making the clearance as small as possible;

commerce, making it as large as possible. If the locks are provided with good controls, electric locomotives for the towing in of ships, sufficient fenders, brake blocks, and the like, the clearance may be kept within average limits. Of course, the possible increase in size of ships should be borne in mind. But if the greatest width of the ship to be used in inland shipping regions is established, it is uneconomical to have too great a lateral clearance. If a wide entrance is desired for special precautionary measures, the lock should have a trumpet-shaped form along both sides. In that case, a train lock of 225 m. (738 ft.) length in the clear should be widened along both sides beginning 25 m. (82 ft.) from the ends. If the greatest ship in question is 10.5 m. (34.5 ft.) wide, inclusive of washboards, the middle section of at least 150 m. (492 ft.) length need be only 11.2 m. (36.7 ft.) wide, but should be broadened to 12 m. (39 ft.) width at the ends. The same results are obtainable by building suitable guide works. In order to facilitate conducting the water out of the lock, a somewhat larger depth clearance is of value. This will not increase the cost of operation. The present 12 m. (39 ft.) width used universally in Germany has doubtless been chosen after consideration of the probable future increases in the size of ships.

The elevation in the clear of superstructures, such as bridges, lift gates, segment gates, and the like, must be that which is customary along canals. In modern German inland ship canals, a clearance of 4 m. (13 ft.) above the highest navigable water level has been adopted; 4.5 m. (14.8 ft.) would have been a better minimum.

D. DESIGN COMPUTATIONS FOR LOCKS AND DOCKS

a. Magnitude of Operative Forces

The design computations of locks and dry docks simply consist in determining the stresses in trough-shaped structures which are subject to excess pressure on one side. The excess pressure on a lock may be directed either inward (common case) or outward. The effect of buoyancy is also of importance. The magnitude of the buoyancy, that is, of the hydrostatic uplift, is equal to the pressure exerted by a column of water having a height equal to the vertical distance from the point under consideration to the water surface. If the wetted surface of a structure (for example, the underside of the floor of a lock) is connected with the water column by conduits of more than capillary size, complete buoyancy is encountered. If the subsoil consists of pervious material, such as sand or gravel and the like, full hydrostatic uplift should be considered to act against the impervious lock lining. For clayey or argillaceous subsoil in close contact with the structure, the magnitude of

the buoyancy may drop to zero. This seldom occurs. In very porous subsoil complete buoyancy frequently does not occur because the bed is so pervious that the water flows through the bottom more rapidly than it can be supplied through the subsoil. In such cases, the uplift amounts to only a fraction of the static buoyancy. Usually this condition does not result in much variation in the design because the loss in buoyancy is replaced by the passive counterpressure of the subsoil. The conditions are different for dock walls. Here the water pressure, which corresponds to the buoyancy on the bottom, plays a very important part. A wall may often be but slightly loaded by the pressure of the earth, but when the water pressure is added the stress may be doubled. Full hydrostatic pressure must be considered on walls if coarse-grained soil is involved.

In order to design these structures properly, an accurate knowledge of the magnitude and nature of the earth pressures is required. The various methods of computing earth pressure are not to be treated here. Accurate investigations should be made of the properties of the soil encountered, rather than merely to introduce mathematical improvement of earth pressure formulas. Improvements in the mathematical treatment of earth pressures are of no value, if, as frequently occurs, mistakes of tenfold magnitude are made in determining the angle of repose. (Instead of assuming a natural slope angle of 28 degrees, one of 33 degrees, etc. is chosen.) It is recommended, therefore, that the following table be used with the Coulomb method to determine the earth pressures for all normal cases of horizontal surface and approximately vertical walls. The active earth pressures in the table are verified by the experiments of Mueller-Breslau, and the passive pressures by experiments of the author.[1] In this particular case, it is advisable to apply from the table the smaller passive pressures given by Coulomb rather than the experimental results of the author. Movement of the wall, which, according to experiments, is necessary to attain the passive pressure of the earth, can not occur in locks and dry docks. To be sure, Coulomb's value will be exceeded, but the amount it is exceeded can not be determined.

Particular attention should be given to the fact that for soil having large pores, such as coarse sand or gravel, the earth pressure and water pressure may be treated separately. Fine sand or sticky soil, on the other hand, retains the water because of inner (capillary) frictional forces. The water is compelled to move with the soil when the wall moves; that is, it is required to undergo the movement experienced by

[1] Müller-Breslau, *Erddruck auf Stützmauern*, Kröner, 1906, Stuttgart and Leipzig; O. Franzius, *Versuche mit passivem Erddruck*, *Bauing.*, Vol. 10, 1924.

TABLE NO. 4

ABBREVIATED EARTH PRESSURE TABLE. LIMITING VALUES OF THE HORIZONTAL EARTH PRESSURES FOR A SMOOTH VERTICAL WALL BY THE FORMULA

$$E=\frac{1}{2}\gamma_e h^2 \tan^2\left(45\pm\frac{\rho}{2}\right)=\gamma_e \tan^2\left(45\pm\frac{\rho}{2}\right)\cdot W = \text{lower wall}$$

Type of Earth	γ_e *Specific gravi'y based upon volume including voids*	*Natural angle of repose ρ in degrees*	$tan^2\left(45-\frac{\rho}{2}\right)$	$tan^2\left(45+\frac{\rho}{2}\right)$	$E_a=\gamma_e tan^2\left(45-\frac{\rho}{2}\right)W=\mu_a W$	$E_p=\gamma_e tan^2\left(45+\frac{\rho}{2}\right)W=\mu_p W$	*Approximate Values* E_a	E_p	*Remarks*
Dry Embankment Earth....	1.4	40	0.22	4.60	0.31	6.4	1/3	6	Using known values of ρ, the minimum values were chosen which give E_a maximum and E_p minimum. According to experiments of the author, the passive values (earth resistance) are greater as a result of the packing of the originally loose earth and an increase in ρ. The experiments were performed with a container measuring 2 m. high and 2 m. wide on the inside. The values given here are particularly useful in making estimates.
Wet Embankment Earth....	1.65	30	0.33	3.00	0.53	5.0	1/2	5	
Dry Clay................	1.6	40	0.22	4.60	0.35	7.3	1/3	7	
Wet Clay................	2.0	20	0.49	2.04	0.98	4.08	1	4	
Dry Sand................	1.6	31	0.32	3.12	0.51	5.0	1/2	5	
Moist Sand..............	1.8	40	0.22	4.60	0.4	8.3	2/5	8	
Wet Sand................	2.1	29	0.35	2.88	0.74	6.0	3/4	6	
Wet Gravel..............	1.86	25	0.41	2.46	0.76	4.6	3/4	4.5	
Sand under water deducting buoyancy and horizontal water pressure..........	2.1−1 =1.1	25	0.41	2.46	0.45	2.7	1/2	2.5	

the earth-pressure wedge. Hence, only a part of the water pressure is free; the water merely functions as weight in so far as it fills the pores of the sand. For large pores and slow displacement of the wall, the water may behave as though it were free. Then the earth pressure, deducting buoyancy, is to be treated separately, using the shear plane observed under the water, and the water pressure is also to be treated separately. As shown by the table, the sum of these two forces is much greater than that indicated for sand filled with water. The differentiation is in accord with natural laws inasmuch as the water in coarse sand moves on its own shear plane, that is, on the horizontal, whereas for the more finely divided earth it moves with the earth on an inclined shear plane.

The design of lock structures begins with ascertaining the external forces; thereafter, the internal stresses of the structure can be computed. The external forces are calculated by consideration of the pressure of the water and of the earth, the weight of masonry walls, the weight of the water within the chamber, and in case of empty dry-docks, the weight of a ship in dock. All forces, including the earth pressure from the sides and weight from above, may be readily determined; the same is true of hydrostatic uplift below the floor and the magnitude (but not the form) of the counterpressure of the soil acting from below. However, the correct determination of the distribution or form of diagram of the bed pressure is particularly difficult.

b. Distribution of Counterpressure on the Base of a Lock

It is difficult to determine the distribution of the counterpressure on the base of a lock. Solving this problem, which is practically insurmountable, is the only difficulty in the exact computation of stresses in the trough-shaped structure. The nature of the forces requires little explanation. All forces acting vertically from above must be counteracted by equal and opposite forces from below; that is, the sum of the vertical forces acting on the structure must be equal to zero. It follows that the counterpressure under the lock floor must be equal to the difference of the sum of the weight of lock structure and water within (or weight of a ship in a dry-dock) and the hydrostatic uplift. Likewise, the resultant horizontal pressure on the lock bottom must be equal to the sum of the pressures of the earth and of the water acting on the outside of the wall; in case the lock is filled, this value is decreased by an amount equal to the horizontal inner water pressure.

If G_o is the sum of the downward forces inclusive of the weight of the bottom, and if A is the hydrostatic uplift, then the counterpressure on base is $G_u = G_o - A$. The forces G_o also include the vertical compo-

nents of the earth pressure E_a, and the weight of the water content of the lock (or dock). These perpendicular forces are dependent upon the movement of the structure. The body of the lock moves downward while being filled, upward when being emptied. Ordinarily, these displacements are so slight that they are without practical importance, but they may become great enough so that the vertical components of the earth pressure must be assumed accordingly. On filling, hence sinking of the structure, the vertical component may be assumed to act upward; on emptying, in a downward direction. The author's investigations indicate that sand and gravel are to be considered elastic substances which behave somewhat like rubber.[1] If $E_a + W_a$ is the external pressure of the earth and water, W_i the inner horizontal pressure of the water, then the horizontal pressure on the lock bottom is $H = E_a + W_a - W_i$. It is necessary to find the position of the line of pressure, especially the altitude of the resultant force H.

The intensity of the counterpressure on the floor is closely related to the magnitude of movement of the bed. In case the soil is not pulplike, the necessary base pressure of the soil is produced by a compression of the subsoil, hence the sinking of the dock when it is filled. If the body of the lock undergoes appreciable distortion under the influence of the outer forces, as occurs with thin concrete bottoms, the distribution of pressure against the base will be dependent upon the amount of the distortion. For example, if the bottom bends downward at the ends, the latter will be impressed deeper into the soil, the middle, less. The total compressive deformation of the soil must average approximately the same regardless of whether the bottom assumes a concave or convex form, or remains flat. However, the intensity of pressure varies from point to point along the bed. A dock structure in which the sides of the floor turn downward will experience greater counterpressure at the sides of the floor than it will in the middle. In case of an opposite distortion, the condition is just the reverse.

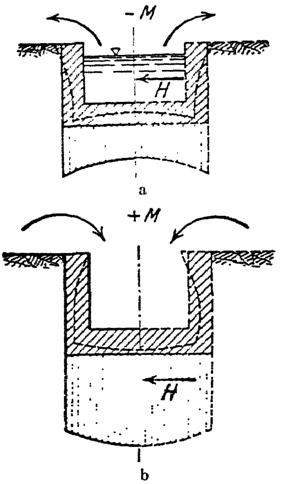
The sum of the horizontal pressures in the floor of the dock may be represented by the force H. The position of this force, H, corresponding to the stress distribution, is dependent upon the distortion of the bottom[2] (Figs. 340 a and b).

Accordingly, the distortion of the floor determines: first, the distribution of the counterpressure below the floor; and second, the position of force, H, with reference to the neutral axis of the bottom. That is, the structure is statically indeterminate at least to the first degree. The calculation is possible if the law governing the relation

[1] *Z. Bauw.* 1908.

[2] Conversely the distortion of the bottom is dependent upon stresses.

between the counterpressures and the line of distortion is known.



Figs. 340 a and b. Distortion and floor pressure

For example, if it could be assumed that the pressure against the bottom is proportional to the perpendicular distance of the line of distortion from a given horizontal, the distribution of the pressure could be readily determined. Such a solution is presented in a doctorate dissertation by Freund.[1] Computation by this method is recommendable for larger structures, particularly for thin reinforced concrete floors which undergo rather large distortions. In hydraulic structures having thick floors, however, the distortion of the bottom is so slight that there is not a great amount of variation in concentration of the counterpressure against the base. For these it may be assumed that the counterpressure is uniformly distributed over the bottom. The justifiable limit of this assumption is treated in a subsequent discussion.

c. The Effect of the Inside Water Pressure upon the Bending Diagram

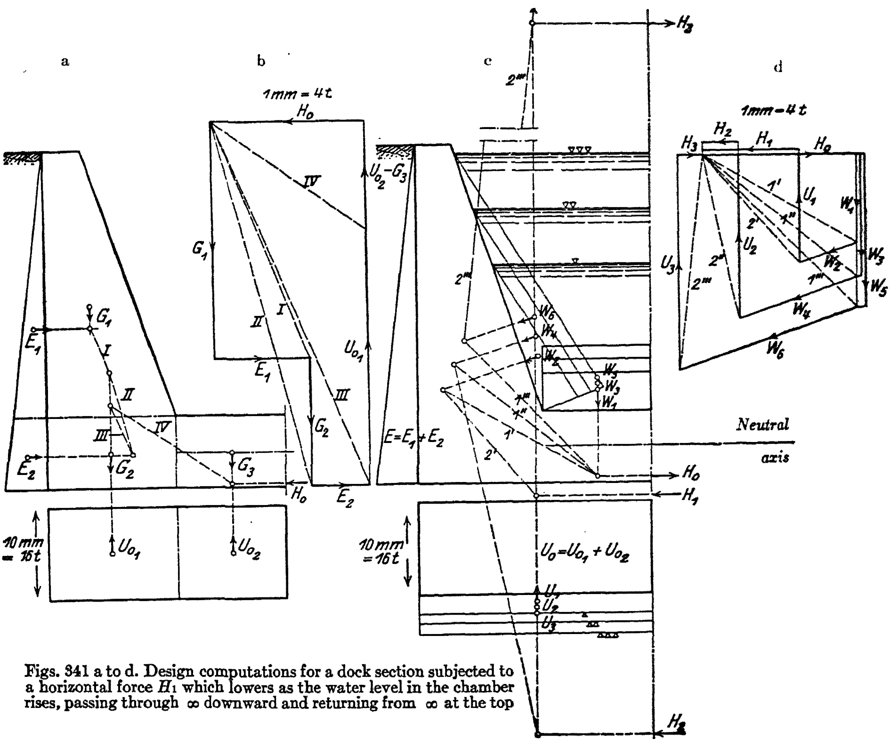
The following is here presented: First, computations for stresses in a dock when empty; second, computations for several different water levels within the chamber.

It is assumed in this calculation that, when the dock is completely full, the water pressure from the inside is greater than the outside pressure against the walls, so that the force H is reversed.

A graphical computation is simple since only the position of force, H, is to be determined (Figs. 341 a to b). The analysis of an empty dock (lock) is made referring to Figs. 418 a and b. The force H_0 lies below the neutral axis. Investigations for three different elevations of water surface are made next in order (Figs. 341 c and d). The magnitude of H decreases greatly from that when the lock is empty but still remains positive, the forces acting from without being greater than the inner water pressures. This second investigation for the position of H is made using the position and magnitude of the force H_0 for the basin empty. The forces due to the inside water pressure are simply combined with the first force H_0, thus giving the positions of the horizontal resultants H_1 and H_2.

[1] Alb. Freund, *Anwendung der Elastizitätstheorie auf elastisch gestützte Körper usw.* 1913, Ernst & Son, Berlin.

The last condition is that for the tank entirely full. For this condition, the water pressure from within exceeds the inward pressures and H_3 becomes negative. Although the next to the last resultant $2''$ is formed by the intersection of H_2 with the vertical counterpressure U_2 below the base, the intersection of the U_3 with H_3 lies above. For an elevation of the water surface causing $H=0$, the resultant of all forces is vertical and the point of intersection of the base pressure with the horizontal resultant ($H=0$) is therefore at infinity. The moment is



Figs. 341 a to d. Design computations for a dock section subjected to a horizontal force H_1 which lowers as the water level in the chamber rises, passing through ∞ downward and returning from ∞ at the top

then $M=0\cdot\infty$, and has a finite value. The algebraic sign of the moment is the same for all cases. In the first three cases M is $(-h)\cdot(+H)$, in the last case $(+h_3)\cdot(-H_3)$, hence negative for all. By applying the moments for a number of elevations of the inside water surface, the magnitude of the moment for $H=0$ can easily be found graphically. In the case discussed, the following change of position of H resulted: as the dock is filled, H drops lower and lower, until inner and outer forces are equal; that is, until $H=E_a+W_a-W_i=0$ and H lies in infinity; however, the stress does not become equal to 0 when the water

surface reaches this elevation. On filling the basin still further, H becomes negative and returns from infinity on the upper side, moving downward as a negative force; during the filling process the stresses in the bottom continually increase. The important factor to note is that on reversing the sign of H the distance of the force from the neutral axis of the bottom passes through ∞.

A uniform variation of the position of H, as described above, does not always occur. Curious circumstances frequently arise in the course of development of pressure curves. For example, in some instances H sinks gradually as the lock chamber is filled, and before attaining the value 0 abruptly returns to the other side where it moves toward ∞ and reappears from the opposite side (Fig. 342 a). For this case h and M do not pass through the 0-point of the H-curve. It is advisable to represent the changes by force diagrams showing all factors in the same diagram, the force H, its lever arm h, and the moment resulting therefrom. A sketch of this nature is shown in Figs. 342 a to d for a standard lock in which the direction of H finally reverses. With the position of force H above the 0-line, the lever arm h is considered positive; likewise, H is considered positive when the excess force is from without. Moments are taken as positive[1] when they distort the lock walls inward.

Fig. 343 shows the effect under the influence of various ground-water levels. Similar results are obtained by assuming various inclinations of the earth pressure acting on the wall. The earth pressure for the same height of water in the dock may be inclined upward or downward depending upon whether the dock is being filled or emptied. The same inside water elevations may cause two altogether different stress conditions in the bottom, although the magnitude of force H is unchanged.

Additional figures show the change of the position of H and accordingly of the stresses when the distribution of the pressure at the base is varied. The uplift due to buoyancy is uniformly distributed in all cases. However, the excess pressure $G_u = G_o - A$ may be variously distributed depending upon the amount of distortion in the floor. Careful attention should be paid to the fact that a logical form of pressure distribution of G_u should be chosen. If the bed is distorted downward at the ends, the force G_u is also greater along the ends of the lock than in the middle, but not *vice versa.* The greatest variation in distribution of force G_u is indicated by a triangle. Since earth pressure always presupposes contact with the earth, it is not possible for the force G_u to become 0 in the middle of the bed. This condition, therefore, must be

[1] In certain cases the greatest stresses occur shortly before attaining the highest or lowest water levels. Compare "Über die Berechnung von Trockendocks," *Z. Bauw.* 1908.

regarded as a limiting condition, just as the other limiting condition assumes uniform distribution of pressure.

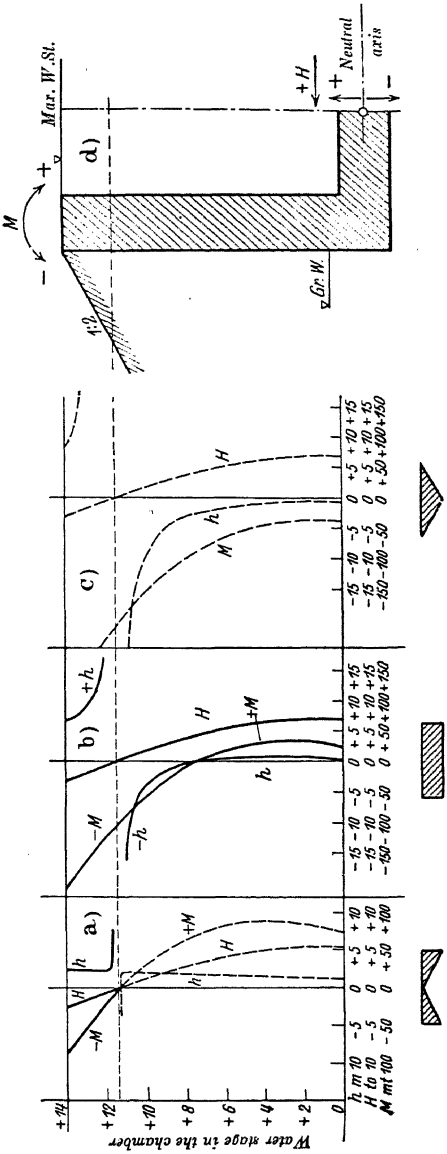


Fig. 342. Horizontal force H, lever arm h, and moment M in a lock subject to various conditions of pressure distribution and various heights of the water level within the lock

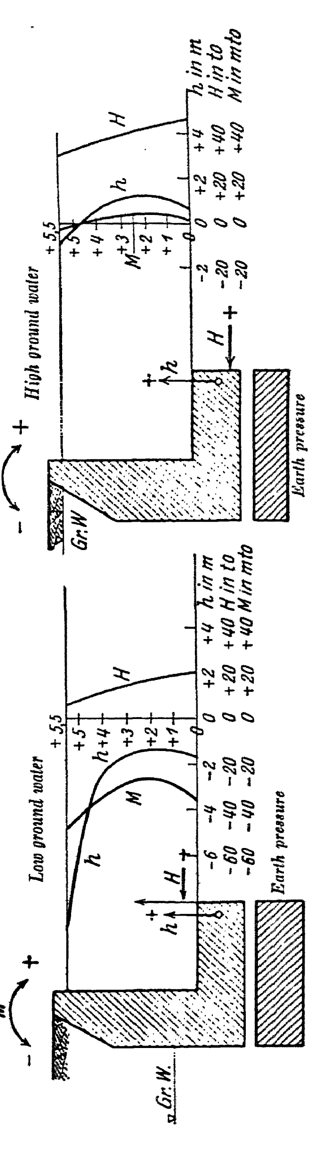


Fig. 343. Horizontal force H, lever arm h, and moment M, for low and high ground water stages respectively. Force diagrams for various elevations of the water level within the basin, earth pressure being considered horizontal

As can be seen more clearly from the investigations of maximum stresses in the floor of the lock, negative bending moments can occur

only if walls are bent outward and the base pressure is similar to that in Fig. 342 a, in which case the uniformly distributed base pressure (Fig. 342 b) may be considered as the limiting condition. For walls bent inward and a base pressure corresponding to Fig. 342 c, a negative moment can not occur. Therefore, the positive moments found in Fig. 342 a are not possible for such a base pressure. But positive as well as negative moments are possible for a uniformly distributed base pressure. The possible moments are drawn in full line; the impossible ones, in broken lines. After completing the computation, a summary is available giving the nature of the moments to which the lock is subjected, the contradictory results then being eliminated.

These various moments can then be grouped into one

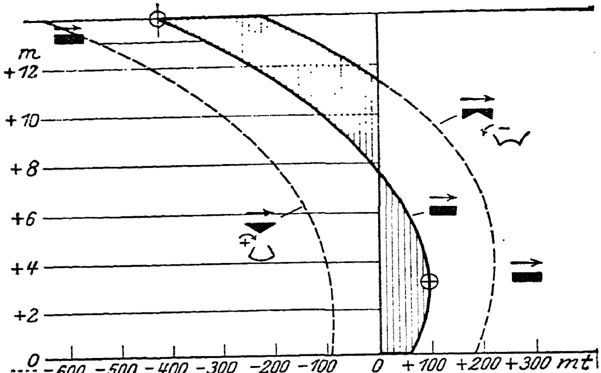


Fig. 344. Moments while filling and emptying a lock, considering horizontal earth pressure and various distributions of pressure against the bottom of the lock

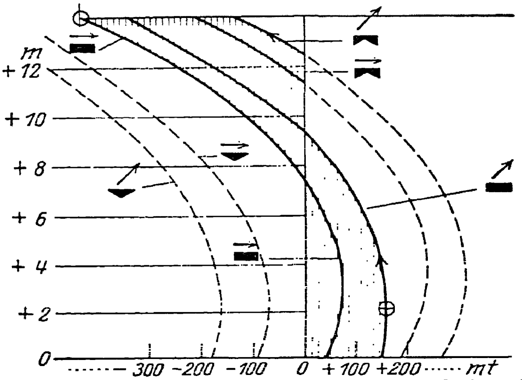


Fig. 345a. Moments during filling and various distributions of pressure against the bottom

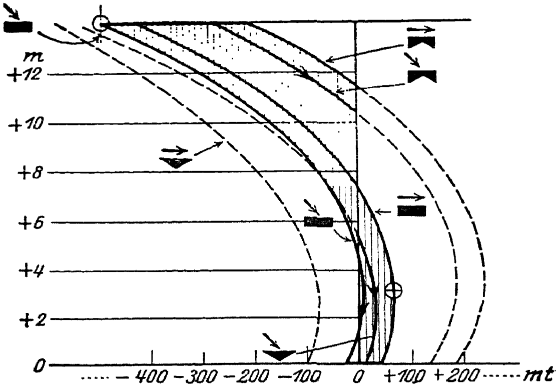


Fig. 345b. Moments during emptying and various distributions of pressure against the bottom

figure, as is indicated in Fig. 344. The pressure from without is considered horizontal. The three moment curves are drawn for the three different base pressure distributions and those moments that are possible, that is, that are compatible with the assumption of the base pressure in question, have been crosshatched. Positive moment is compatible only with uniformly distributed bed pressure or greater pressure in the middle than along the sides. This latter pressure distribution, however, yielded only negative moments; these are impossible. Thus the positive moment does not exceed 100 meter tons, the negative 420 meter tons. The floor should be reinforced for these moments.

Similarly, Figs. 345 a and b indicate the possibilities for a dock which sinks on being filled and rises on being emptied. While being filled, in the case of the dock sinking into the soil, the lateral pressure of the earth on the walls may be exerted horizontally or inclined upward. The moments possible in this case for the various water levels are crosshatched. Similar diagrams have been made for the dock in the process of emptying. Investigations by the author on dock *V* of the Imperial Shipyards at Kiel, indicate that docks may settle as much as 10 mm. (.390 in.) in the sandy subsoil when being filled and again rise by this amount when emptied.

A more exact consideration of the aforementioned diagrams shows clearly that the greatest moments in the middle of the floor slab occur when there is a uniform distribution of the pressure under the slab.

d. Calculation of Maximum Floor Stresses

The majority of failures in lock floors have occurred in the form of longitudinal cracks in the middle of the slab. Other locks and docks had cracks under the inner footing of the wall. Cracks between these two limits are probably the result of nonhomogeneous material.

Consideration will be given to the form of pressure against the base which will cause the greatest stresses in the middle of the floor and under the wall footing. In so doing, the hypothesis as to the agreement between the form of the pressure at the base and the distortion of the slab are to be considered:

(*a*) The case in which the line of pressure lies entirely above the neutral axis.

(*b*) The case in which it lies entirely below the axis.

(*c*) The case in which the line of pressure at the sides lies above the neutral axis but below it in the middle. In the last case, the floor is subject to double bending.

The various cases are treated graphically in Figs. 346 a and b.

In case (*a*) there is only one possible assumption; namely, that the

floor is distorted upwards at the ends (Fig. 346 a). The resulting bed pressure must be greatest in the center. It is possible to imagine a condition in which an originally rather hard clay bottom is softened by the penetration of water, so that its elastic resistance decreases and the material gradually assumes hydraulic properties. As a result the coun-

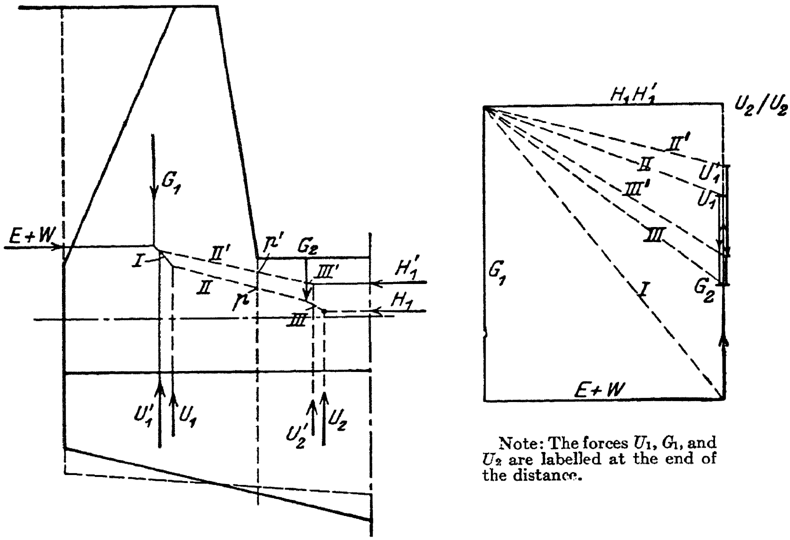


Fig. 346a. Statical investigation of the case in which the floor and sides are bent upward

terpressure in the middle gradually becomes smaller and the entire structure sinks somewhat deeper, during which process the pressure at the ends increases. The pressure is readjusted in the direction of the broken line. Force U in the force diagram has been resolved into the components U_1 and U_2. The resolution of these component forces gives the inclination of the resultant force II, whose point of intersection with the force U_2 determines the altitude of H. If the earth counterpressure becomes uniform, U_1 increases, whereas U_2 decreases in the same amount. The inclination of force II' flattens, H rises. Since H remains constant in magnitude and lies below the zero line, the stresses in the middle of the slab increase as H rises. The point of intersection, p, of the vertical through the wall footing with force II' rises as force II' becomes more nearly horizontal, resulting in an increase in the stresses at this section. The limiting condition is reached when the earth counterpressure assumes a rectangular distribution. From this it follows that for H above the neutral axis, the floor stresses reach a maximum value if the earth counterpressure is uniformly distributed.

Case (*b*) presents the reversed distribution of floor pressures (Fig. 346 b). Working on the assumption of softening the subsoil layer together with equalization of the pressure, the following reasoning may be used. If the pressure becomes more nearly uniform, the original force U_1 becomes smaller. The resultant force *II* becomes steeper, and

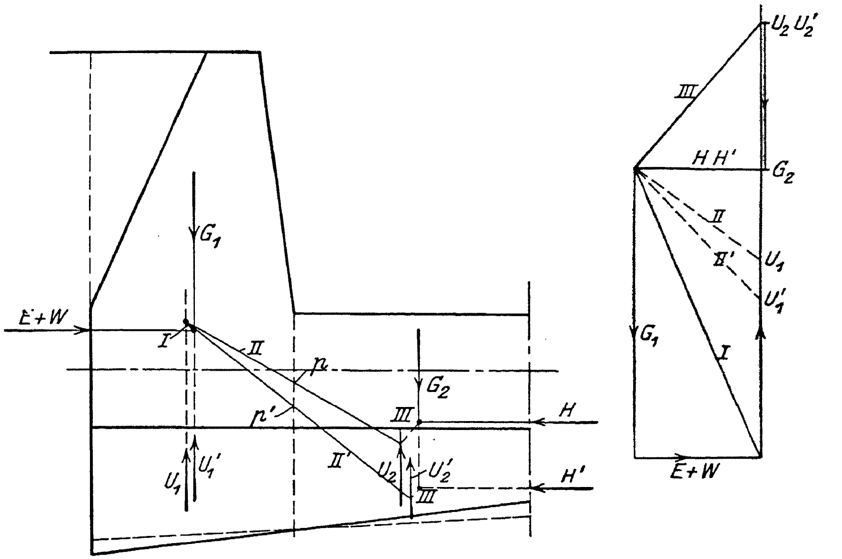


Fig. 346 b. Statical investigation of the case in which the floor and sides are bent downward.

as a consequence point *p* and the resultant force *H* sink lower. Hence, the stresses in both sections increase, provided the magnitude of force *H* remains unchanged. The limiting condition, with the maximum value of *H*, is again reached when the soil pressure below the floor slab becomes uniform.

Thus, for uniform bed distortion the stresses in the floor slab assume the maximum value when the pressure below the floor slab is uniformly distributed. In most dock floors the distortion is so slight, because of the great thickness of the floor slab, that an appreciable variation in the magnitude of the soil pressure at the ends and in the middle of the bed will not ordinarily exist. Therefore, by assuming a uniform distribution of the bed pressure there is certainty of coming very close to reality, and, furthermore, of finding the greatest floor stresses possible.

Case (*c*) does not permit of such a simple solution. For this condition the middle section also receives its greatest stress for the assumption of uniform distribution of bed pressure. The side joint, however, can assume greater stresses for non-uniformly distributed counterpressure.

Nevertheless, if the floor slab is thick the difference in stress cannot be very great. Consequently, the author also recommends assuming a uniformly distributed bed pressure in this case.

The method formerly acceptable for bridge foundations and occasionally recommended for lock floors, is impracticable because it assumes force H in the middle of the floor as acting at the neutral axis and from this determines the distribution of pressure at base. Because the magnitude of force H is completely known from the external forces, only its position with reference to the neutral axis need be determined. If the position of H is presupposed, the problem is considered solved because of an arbitrary assumption, and then the form of the subsoil is wholly immaterial. This method underrates the nature of the problem and should not be used.

e. Detail Computations

In addition to the foregoing analysis, particular parts of the wall that seem especially endangered, such as in the vicinity of shafts through the wall and where several canals above one another are left open in the wall, require individual calculations. Places against which the mitering pressure acts, in case of mitering gates, require special consideration. When reinforced concrete is used, individual forces may be distributed over rather large wall lengths. Shafts are lined, and the walls in the vicinity of shafts are strengthened by longitudinal steel bars so that the load on the hollow section also acts along a greater length of the wall.

Particular difficulties arise in computing the lock cross-sections if the floor is divided into three longitudinal strips by longitudinal sheet piling at the base of the wall, as formerly had been proposed for the dry docks of the German Imperial Marine. For such a design the walls must first be calculated separately and the floor separately. If the structure is erected in the dry with a lowering of the ground water, then a change in conditions for the wall occurs when the dock is filled. (The reverse is true if the structure is built with concrete poured under water.) The resulting action is very difficult to determine, and it is necessary to resort to trial computations in order to approach actual circumstances.

The following simplified computation (Fig. 347) is recommended for computing the stresses in a wall containing a large culvert in the lower part. The wall is computed as far as a horizontal section which lies at the crest of the culvert. Next, the stress diagram, which ordinarily is in the form of a trapezoid, is sketched. Suitable stress trapezoids are then distributed from the crest to both sides and the distributed load replaced by two resultants. The left and the right parts of the wall

(Fig. 347) are then analyzed as though they were independent units. Below the canal the force components are again brought together as one resultant force. This method of computation neglects the fact that the wall around the culvert forms a complete frame. The calculation, therefore, exaggerates conditions. It is very simple, however, and on the side of safety. Calculating the walls by the methods used for full-portal iron structures can not be used successfully; the force distribution will show itself entirely differently than would be the case with slender steel bars. The simplified method is best in this instance.

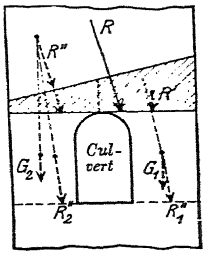


Fig. 347. Statical investigation of a cross-section of the wall which is weakened by the insertion of a culvert.

Because of the uncertainty of all assumptions and the great cost of such structures, it is advisable to make numerous trial computations. All possible combinations of circumstances that might conceivably develop should be investigated. The amount expended in these computations is so slight in comparison to the cost of the structure that the money consumed in such work is well invested. Furthermore, investigation of the probable stressed conditions will pay for itself, either through a feeling of assurance, or through actual savings in the construction cost. Special design precautions are invariably advisable because of the significant number of lock and dock failures.

Computations for the design of lock structures can be made by purely numerical methods. After establishing the distribution of the subsoil pressure, the numerical method offers no difficulties whatever, The graphical presentation is preferable because it permits the eye to follow the lines of the forces. In final designs which are to be executed it is well to check the graphical methods by numerical ones. Care should be taken that no forces are neglected and that forces which are apparently neutralized in that they act on both sides of the wall be entered regardless of this fact, so as to determine the pressure curve, because before the forces are neutralized they must exert pressures in the masonry.

E. EQUIPMENT FOR FILLING AND EMPTYING

a. Various Arrangements for Filling and Emptying Locks

Leonardo's invention of building valves in the lock gates was retained for centuries as the sole arrangement for filling and emptying locks as well as dry docks. It was entirely adequate for small ships but became unsatisfactory as the size of ships increased. For example, the Finow Canal locks, which are 41.0 m. × 5.3 m. (134.5 ft. × 17.4 ft.) in plan

and have about a 3 m. (10 ft.) lift, required only 660 cu. m. (23,360 cu. ft.) of water per operation, while 40,000 cu. m. (1,416,000 cu. ft.) and over is required in modern locks. Gate sluices (sluices in the lock gate) have been maintained up to the present day as reserve arrangements, but the tradition of building them into the lock gates as reserves should be discarded. It would be preferable to reintroduce them as main valves.

A single point of view predominates in passing judgment upon a filling or emptying appliance. It is simply that the water must flow into and out of the lock in such a manner that the ships lie quiet within the lock. It is not possible to fill and empty a lock without some surging inside the lock chamber, but there is a basic difference between filling and emptying the chamber. During the latter period, shortly before complete emptying, there is only a very slight difference in pressure between headwater and tail-water, so that the gates may be set wide open; the water flows relatively calmly. At the beginning of emptying there is a deep water cushion under the ship. Consequently, the motion in the chamber at this time, for a given valve opening, is less injurious than during filling. When the lock is being filled, the ship at first floats on the tail-water; that is, with very little water beneath it. If the valves, which are subject to the same excess pressure that they were when the emptying began, are now opened as widely as they were during the initial emptying, the influence upon the ship is much greater because of the shallow depth of water in which the ship floats. It is imperative, therefore, to fill the lock more slowly than it may be emptied.

A number of figures are presented to show various types of filling and emptying appliances. Fig. 348 shows the original arrangement for filling through gate valves. The water itself generates a current in the chamber; the current in the lock chamber is directed from the headwater to the tail-water.

Fig. 349 shows the arrangement for a simple by-pass culvert. Culverts or sluiceways can be closed by valves or sluice gates built into the walls at the ends of the lock gates. Originally, simple sliding valves patterned after those used in lock gates were introduced, but far-reaching developments have since taken place. The movement of the water is the same as it is in case of gate valves. The use of culverts has an advantage over valves in the lock gate, for if they are directed toward each other, in the upper end of the lock (Fig. 349), the water jets impinge against each other in front of the lock and thus destroy their energy. For the lower bay the arrangement results in the same current in the outer lock as with gate valves (valves in the lock gates). Accordingly, this type of culvert (without spur culverts) offers no advantage over

openings in the lock gates. In case of steel mitering gates, valves should be built into the lower gate only. In steel structures this is readily accomplished without weakening the gate. The valves may be made to extend over the entire breadth of the gate.

The first locks with culverts were built in Holland long before 1600 Figs. 351 a and b show the further developments of this principle In these layouts the by-passes were built throughout the wall area of the lock. Many small spur openings enter directly from the main culvert to the floor surface. Flow is cut off by closing one end of the culvert. The longitudinal current in a lock of this nature is inappreciable, but strong cross currents are set up in its place. The valves require attention more frequently than is necessary when they are in the gate, because troublesome cross currents might otherwise occur, tending to throw the ship against the wall of the lock. The column of water between the ship and one wall may even be drawn off so abruptly that the ship is forcefully jammed against the wall by the excess pressure on the other side.

The spur conduits are spaced at uniform intervals. Theoretically, the distances should decrease toward one side. When the lock is being filled, the distance through the culvert to the spur conduits near the tail-bay is greater than it is to those near the head-bay. Accordingly, the spurs at the tail-bay should be spaced at shorter intervals. On being emptied, the contrary is true; the distance from the spurs to the tail-water is shorter near the lower end; therefore, the spur conduits at the upper end should be spaced at shorter intervals. Both conditions are neutralized because the culvert must of necessity maintain the same cross-sections for the full length, since it must operate both during filling and emptying. Experience indicates the system to work most effectively with uniform spacing of the spurs, but in such a layout, longitudinal currents as well as cross currents will occur in the chamber, because the efficiency of the spur conduits is different for every point of the lock. In inland ship locks these spur conduits are set at an interval of from 6 to 12 m. (from 20 to 39 ft.). The mouths of the spurs lie directly across from one another. Displacing them would cause vortex currents.

The most ideal but also most expensive solution is procured by arranging spur culverts in the floor transverse from the main culvert, and constructing spur conduits vertically from the lower spur culverts, instead of permitting the spurs to empty directly into the chamber from the main culverts. In the ideal layout the water rises vertically when the chamber is being filled and falls vertically when being emptied. Harmful blows against the lateral walls of the vessel are practically eliminated thereby. Fig. 351b shows an arrangement of this nature as

used in the Panama Canal. Even though the valves are opened very unevenly, incoming water is uniformly distributed and currents are practically absent. In fact, the lock may be filled by flow through one side culvert while the other remains closed. Fig. 366 shows the arrangement of the culvert system of the Panama Canal locks in more detail, but the cylindrical main culvert valves are not indicated. An arrangement almost as good as this one can be obtained by laying the longitudinal culverts in the bed instead of in the masonry walls, and providing for the spur conduits to project upward from two or three parallel culverts. The lock from Lake Superior to Lake Huron at Sault Ste. Marie is built according to this system (Fig. 350 b). The closure is effected above by horizontal revolving valves.

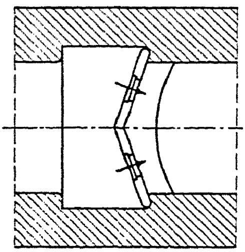


Fig. 348

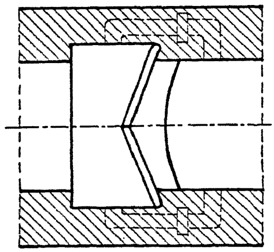


Fig. 349

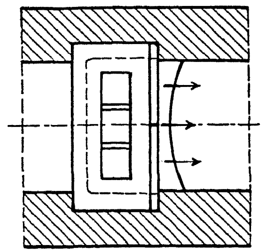


Fig. 350 a

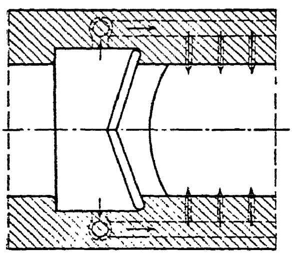


Fig. 351 a

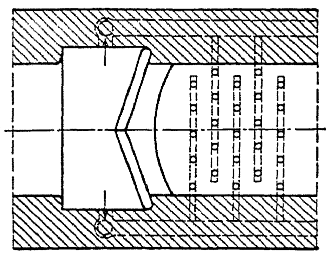


Fig. 351 b

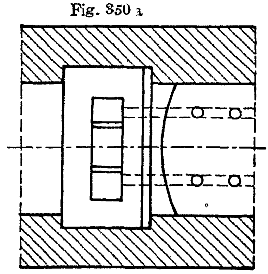


Fig. 350 b

Figs. 348–351b. Various arrangements for filling and emptying a lock chamber

Fig. 348. Valves in the lock gates
Fig. 349. Simple culvert around the side of the gate
Fig. 350a. By-pass of the stilling basin (by Mohr)
Fig. 350b. Vertical gate by-passes. Longitudinal culvert in the floor
Fig. 350c. Side drop shafts with stilling basin
Fig. 351a. Culverts with spur conduits
Fig. 351b. By-passes with spur culverts under the floor

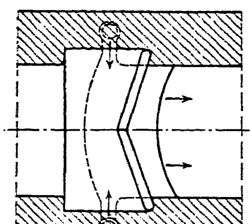


Fig. 350 c

The great cost of the long culverts has recently contributed to a return to use of short by-passes around the quoins of the lock gates. The large new train lock at Hemelingen, Germany, for example, contains the short by-passes, the longitudinal current in the lock being tolerated to decrease the cost. Investigations have

been directed towards making the longitudinal currents as harmless for ships as possible. The work of Mohr (builder of the Oder-Spree Canal) and of Krey (the late head of the Prussian Experimental Station of Hydraulics and Shipbuilding in Berlin) are noteworthy in this connection. Mohr equipped the locks with a large relief basin under the upper end because he realized that filling is of far greater danger to the ship than emptying (Fig. 350 a).[1] The water falls through horizontal valves into a stilling chamber where it loses most of its energy and then flows through the lock in a fairly uniform stream. Krey investigated the use of segment or lift gates which were to serve both filling and emptying the lock. Here also the water was to fall over the top of the gate into the lock and then flow through the entire chamber. The procedure causes difficulties in canal locks if the gate is to be raised, because the water must then fall abruptly over the spillway into the lock chamber. The solution proposed by the author for the canalization of the Leine River eliminates this disadvantage. Here the water was to be conducted to a stilling basin through a slot in the floor at the head-bay; from there it was to enter the lock chamber as in Mohr's plan. The use of lift gates at the tail-bay introduces some difficulties if the gate heights are very great, but these shortcomings are not insurmountable. A good solution consists in emptying the lock under the gate.

Particular attention should be given to preventing the water from drawing air as it flows into the culvert. Regardless of whether siphons, horizontal culverts, or any other type of conduits are considered, the inflow of air should be avoided because this greatly influences the efficiency of the conduit. Important experiments in this connection on the head-bay of the locks at Anderten (Midland Canal, Hanover) were made by Krey. These experiments led to an original method of regulating the intake at the head-bay. Fig. 359 shows a cross-section through the head-bay of the works drawn according to the original proposal. Each by-pass was to have a special entrance into the side wall. The experiments showed this arrangement to be unsuitable. Finally the solution represented in Fig. 360 was evolved. In this design the water does not enter the shaft of the bed from the side but from immediately above, similar to the arrangement in the locks by Mohr, along the Oder-Spree Canal, at Sault Ste. Marie, etc. As may be seen from the figure, the shaft at Anderten has a special form. The figure should not be imitated mechanically for other lifts; in case of locks having significant differences in dimensions from this one, new experiments should be performed.

Especial difficulties arise during filling and emptying locks with

[1] Dr.-Ing Burkhardt, Schleusen ohne Umläufe, *Baut.* 27, Vol. 3.

thrift basins which are to be described later. For the present, attention is called merely to the fact that in operation much depends upon obtaining a well-regulated flow of water in the lock during filling and emptying. The structure should be so designed that the currents persist in the one direction and reverse currents are avoided, so that ships are always forced in the same direction by the current and lie quietly in their hawsers. The same holds true for culverts. If the current is reversed along individual stretches, as may readily occur in open parallel-connected thrift basins, the efficiency of the by-passes is correspondingly diminished.

An excellent solution for a lock with spur canals was obtained in the thrift (storage basin) lock at Minden. The lock has a lift of 15 m. (49 ft.). The ships within lie almost motionless during a filling period of only seven minutes. At Datteln (Heinrichburg, Germany) an otherwise highly successful layout, consisting of a shaft lock of the same height with open thrift basins, is found unsatisfactory in this respect. The shortcomings were due to the open thrift basins which cause irregular flow in the lock. The time required for filling is almost twice that at Minden.

b. The Time Necessary for Filling and Emptying

The time required for filling or emptying a lock is but a portion of the locking time which a ship requires to pass the lock, and for the most part it is the smallest portion. The layover at the lock can be estimated at from twenty minutes to one hour depending upon the size of the lock. The time for filling and emptying in such cases usually amounts to only five or ten minutes. Nevertheless, the calculation of the filling time is very necessary because the assumed periods might very easily be greatly exceeded if the filling equipment is not properly gaged.

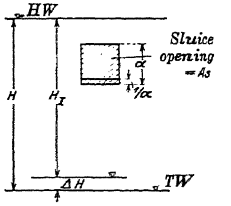


Fig. 352. Sketch concerning time required for filling. A_k=surface area of chamber. A_s=effective cross-sectional area of valve. H= pressure head at the beginning of the filling operation. $\frac{1}{\alpha}A_s$= effective valve opening at the beginning of the filling operation

The simple formula for the discharge from a reservoir is employed in this calculation. For a pressure head, H, the velocity of flow from the tank is

$$v=\mu\sqrt{2g\,H},$$

in which the result depends upon the determination of the coefficient μ. The value μ may be estimated as follows:

Kind of Inflow	μ
Valves directly in lock gates	0.8 to 0.6
Short culverts around gates	0.7 to 0.5
Long wall culvert	0.4

For wall culverts with spur outlets, μ must be computed by pipe formulas. The value given above is useful only for rough estimates. Bazin's formula is recommended for this purpose. The surface area of the chamber is designated by A_k, the sum of the valve cross-sectional areas by A_s, the pressure head when filling begins by H, and the valve opening by $(1/\alpha)A_s$. If the valve maintains this position until the water has risen a height $\Delta_1 H$ so that the pressure head $H_1 = H - \Delta_1 H$ (Fig. 352), then the rate of flow at the beginning of the period will be

$$\frac{1}{\alpha} \cdot A_s \cdot \mu\sqrt{2g\,H},$$

and at the end of the period,

$$\frac{1}{\alpha} \cdot A_s \cdot \mu\sqrt{2g\,H_1}.$$

The average rate for the differential period, therefore, will be approximately

$$\frac{A_s}{2\alpha}\mu\sqrt{2g}(\sqrt{H}+\sqrt{H_1}).$$

The time required for filling the lock the distance $\Delta_1 H$ is

$$\Delta_1 T = \frac{2\alpha A_k(H-H_1)}{A_s\mu\sqrt{2g}(\sqrt{H}+\sqrt{H_1})} = \frac{2\alpha A_k}{\mu A_s \cdot \sqrt{2g}}(\sqrt{H}-\sqrt{H_1}).$$

For the next period with the valve opening $(1/\beta)A_s$ and a rise in the water height from H_1 to H_2, the following time interval is required:

$$\Delta_2 T = \frac{2\beta A_k}{\mu A_s\sqrt{2g}}(\sqrt{H_1}-\sqrt{H_2}), \text{ etc.}$$

During the last period, the valve is wide open so that

$$\Delta_n T = \frac{2\rho A_k}{\mu A_s\sqrt{2g}}\sqrt{H_n}.$$

If the filling operation is divided into four periods, the total time is

$$T = \Sigma\Delta T = \frac{2A_k}{\mu A_s\sqrt{2g}}[\alpha\sqrt{H}-(\alpha-\beta)\sqrt{H_1}-(\beta-\gamma)\sqrt{H_2}-(\gamma-\delta)\sqrt{H_3}].$$

As an example, assume a lock of 12 m. lift and that $\mu=0.4$, $H=12$, $H_1=10$, $H_2=8$, $H_3=5$ m., $\alpha=4$, $\beta=3$, $\gamma=2.1$, and $\delta=1$.

Then the time

$$T = \Sigma \Delta T = \frac{1}{0.2\sqrt{20}} \cdot \frac{A_k}{A_s} [4\sqrt{12} - (4-3)\sqrt{10} - (3-2.1)\sqrt{8} - (2.1-1)\sqrt{5}]$$

$$= 1.11 \frac{A_k}{A_s} [13.86 - 3.16 - 2.55 - 2.47]$$

$$= 6.32 \frac{A_k}{A_s}.$$

If a ratio of $A_k/A_s = 100$ is chosen, the time required for filling the lock will be 632 seconds or 10½ minutes. Should the gate have been opened widely at the start, the time would have been

$$T = 1.11 \frac{A_k}{A_s} \cdot 1 \cdot \sqrt{12} = 3.81 \frac{A_k}{A_s} = 381 \text{ seconds or } 6.4 \text{ minutes approximately.}$$

This comparative computation indicates that not much time can be gained by a careless overly rapid opening of the gate. The actual time required for filling will be governed by various circumstances; for example, the sailors must relay the hawsers anew after the ships have been lowered a certain amount. The arrangement of these fastening appliances must be adjusted to a desired rhythm in operation. Slight variations are immaterial.

F. CONSTRUCTION OF FLOORS, WALLS, SILLS, ETC.

a. Floors and Sheet Piling

In designing locks, their historical development should be borne in mind. Strictly speaking, a lock is merely a short stretch of a canal or canalized river in which the weirs are very close together, or it may be considered as a small dock harbor along the sea. In any case, neither a stretch of river nor a harbor bottom has been provided with a concrete bed to produce complete impermeability. Nowadays it often is not necessary to build lock floors in such a manner that the whole chamber resembles a trough. A lock chamber may be constructed of concrete or masonry even though impermeability is not necessary; this should be done because of the current in the lock chamber during filling and emptying, if the substrata are erodible to the extent that serious scour may occur.

The conditions governing the design of the bays of a lock are entirely different from those governing the design of the chamber. The bays are to be considered movable weirs requiring very low base plates for closure. In case the substrata do not consist of bed rock, sheet piling must be driven below the weir. Sheet piling is dispensable as a pre-

cautionary measure against undermining along the chamber, but deep transverse sheet-pile walls at the bays may not be omitted where driving is possible. Sheet piling is driven around the chamber to maintain the construction pit, but not to provide an impermeable obstacle to the flow of ground water.

b. Construction of the Bays of Locks

The most difficult structure of canal and river locks is the head-bay. The floor of the head-bay serves merely to facilitate the transition from the upper canal level to the drop wall, and to render possible a port sill for the gates. In Figs. 353 to 358, six different forms of head-bays are shown. The types in Figs. 353 and 354 are to be considered for rather simple locks constructed on soil which may be compacted. In homo-

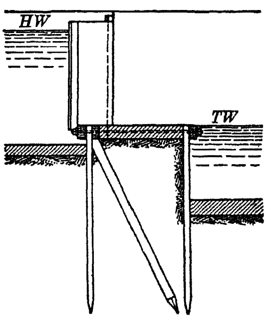


Fig. 353. Development of sill with sheet piling (for a mitering gate)

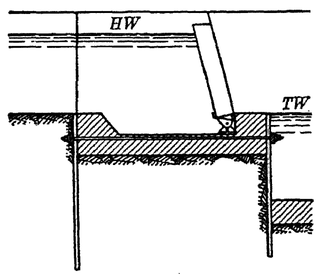


Fig. 354. Development of sill with sheet piling (for a trap gate)

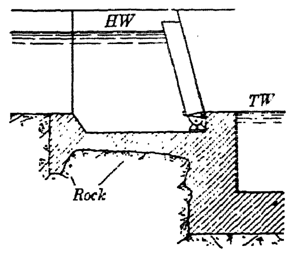


Fig. 355. The masonry construction in case of bed-rock floor

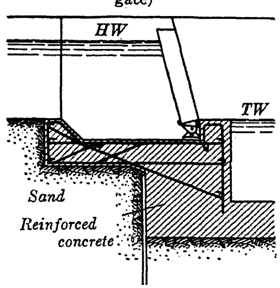


Fig. 356. Reinforced concrete in case of sand or loam substrata

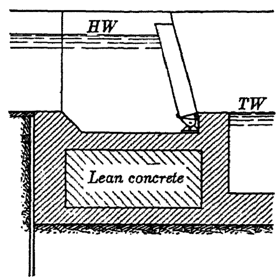


Fig. 357. Masonry in case of sand or loam substrata

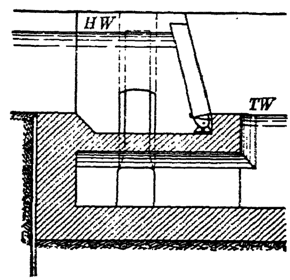


Fig. 358. Arrangement in case of stilling basin

Figs. 353–358. Details of head-bay

geneous rocky substrata the form in Fig. 355 may be used with a thin head-bay floor; that in Fig. 356, on the other hand, with its abrupt offset between the massive part of the bay and the thin floor in the upper level, is statically so unfavorable that it should seldom be built. This form is feasible only when the thin end of the floor is short and connected with the thick block by sufficient reinforcement. The next type

(Fig. 357) is best adapted to rather soft soil, if the soil is so unhomogeneous that it is advisable to lay the upper end just as deep as the chamber walls. In this case a good solution may be obtained by the use of hollow concrete construction. Mohr's method of feeding water to the lock embodies the use of a stilling basin at the upper end. The concrete fill may be omitted, leaving a cavity which is suitable for use as a stilling basin. The final valuable form of head-bay structure is obtained (Fig. 358) in this manner. Figs. 359 and 360 show two different forms of section for the head-bay of a shaft lock (Anderten Lock). The form in Fig. 359 was abandoned as a result of experiments favoring the other in which the water flowed better.

Tail-bays have the upper surface of the floor at the elevation of the upper surface of the chamber floor. The floor should be made only as thick as statically required. In order to minimize the foundation depth, it is frequently advisable to use steel reinforcement. Reinforcement is usually not necessary in the floor of the head-bay, however, because of the great thickness of this part of the structure. No rule can be set up to fit all circumstances, for the design must suit local conditions.

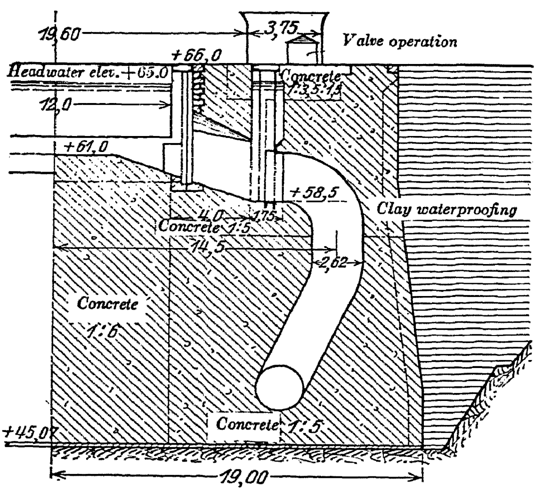
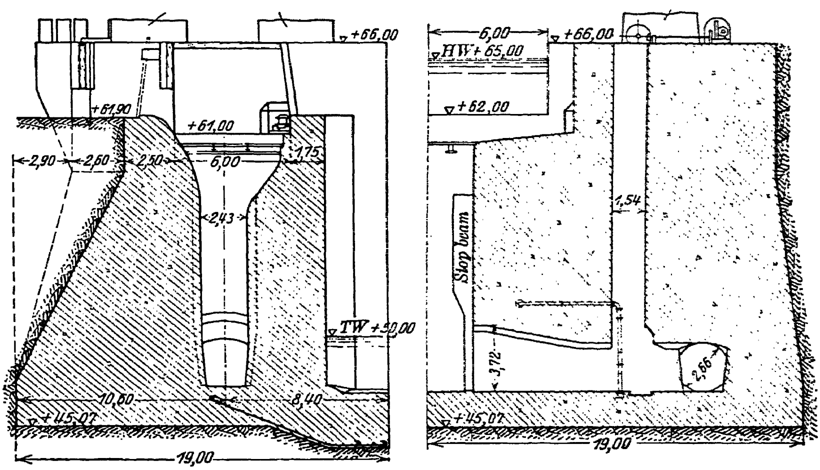


Fig. 359. Lock at Anderten. Design originally proposed

Each bay should be provided with at least one transverse row of sheet piling, preferably with two. The head-bay sheet piling is the more important. The floor of the bays must be built strong enough to cause the end to act as a rigid body in which the walls and floor can not undergo appreciable movement. Rigidity is necessary to assure satisfactory functioning of the lock gates.

The walls of the lock bays are made particularly heavy, especially in the lower part, if by-pass culverts are installed. The culverts assume an extremely large diameter; for example, in large sea locks the by-passes are greater in diameter than railroad tunnels. (The culverts of the Panama Canal locks are over 6 m. (20 ft.) in diameter.) Inasmuch as valve equipment must be placed in shafts, the wall

at this place must frequently be made about as thick as the wall below. It is usually inadvisable to place the walls of the bays independently of the floor. Preferably a thick, rigid base should be built with adequate reinforcement, the walls being superimposed. The bays of sea locks form an exception to this procedure when the floors are as much as 5 to 8 m. (16 to 26 ft.) thick. For these the walls should preferably be built first and the floor fitted into the locks in a manner preserving the necessary rigidity.



Figs. 360 a and b. Lock at Anderten Revised design of head-bay

The finish of the sill in a bay requires special care. Most classes of gates (segment and cylinder gates excepted) fit against the sill. In small locks the heavy floor may be omitted if the sill is adequate. In this case (Fig. 353) the sill must be replaced by sheet piling supported by batter piles, and the drop wall must be held secure by a second line of sheet piling. This arrangement assumes the form of a cofferdam, but such bays are very rarely constructed. This style of structure would be justifiable in colonial regions, because wooden locks built of the classes of wood found there would be economical.

Fig. 361. Sill stone with spherical bottom

The sill generally must be set with stone or fixed with rolled iron layers or cast steel edges. Intense pressures are exerted upon the sill which must be transferred into the floor at the bay. Hence, the sill must be well anchored and must be so arranged that it is not endangered by buoyancy. In large dry docks, buoyancy is especially dangerous; for example, at 10 m. (33 ft.) depth below water, a buoyancy of 10 tons per sq. m. may occur.

The sill plate should preferably be constructed integrally with the floor so that there is no joint between the two parts. In cases where it is not feasible to lower the ground-water level, the dock is first roughed in and subsequently finished. The concrete is poured between lines of sheet piling in the wet. After the bed is drained, the finished structure is begun. The sill stones are anchored from below in the finished floor. The outer lock, or outer space of the dock, is lined with clinker set in cement up to the elevation of the top of the sill. Preparing, laying, and closing joints of the quarry-stone proceeds in the following manner. Individual stones are prepared not only on the top surface and along the sides, but also on the bottom. Spherical projections are chiseled out on the bottom so that the lowest point lies in the middle of the stone. They are then laid side by side and the joints filled with mortar.

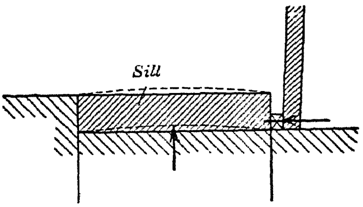


Fig. 362. Bending in sill

Since each stone is given a regular bottom surface with but one very low point, air can not gather beneath the stones (Fig. 361). With the usual procedure of laying stone and filling the joints, the cement frequently penetrates the joint only at the edges, because trapped air prevents it from filling the cavities beneath the rock. The water may then penetrate this space below the stone and develop a buoyancy force. Many sill failures are attributed solely to the improper preparation of the stone.

The pressure of the gate against the sill should act as nearly as possible on the center of the stones so that only normal stresses are generated in the sill plate. If the point of contact lies too low, as indicated in Fig. 362, the sill plate may be lifted from the floor in consequence of the distortion, and lead to removal of the plate from the bed. For locks built in the dry, as in the construction of the Panama Canal locks, the sill and the drop wall should be anchored by iron scaffolding which is concreted into the structure.

c. Construction of Lock Chambers

The lock chamber might be built wholly as a stretch of a canal or harbor. Long barge-train locks may be constructed using side slopes which, together with the bed, can be made secure solely with paving stones, if the rather great loss in water can be borne. A slope of the chamber walls, of course, results in increase of the chamber capacity. This increase is important commercially in so far as it lies above the tail-water. The part below the tail-water surface remains filled and does not enter into consideration as far as water consumption is concerned.

There are three types of chamber walls: (1) the slope, (2) the bulwark, and (3) the masonry wall. The viewpoints which govern the design of walls in tidal regions are also applicable to chamber walls. In tidal

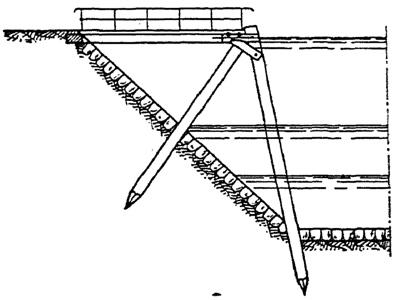


Fig. 363 a.

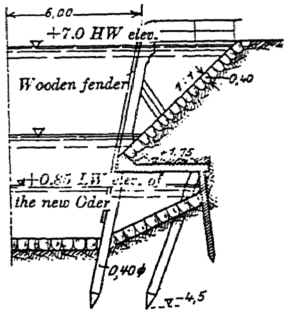


Fig. 363 b

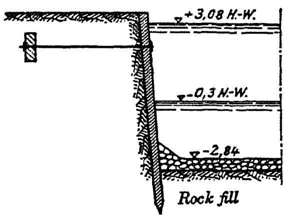


Fig. 364 a.

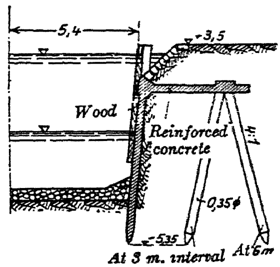


Fig. 364 b.

Figs. 363–364. Designs for lock chambers without masonry floors

Fig. 363 a. Inclined side slope paved with stone
Fig. 364 a. Simple sheet-pile wall
Fig. 363 b. Inclined slope with paving and bulkheads (*Ost oderschleuse*)
Fig. 364 b. Bulkhead (*Westoderschleuse*)

regions the change in stage of the water takes place in approximately six hours, in lock chambers the interval is frequently only six minutes. Thus, the formation of water pits in lock walls is much more likely to occur than in sea walls. Lock walls, therefore, are endangered more than others.

Figs. 363 to 364 show the various forms of chamber construction without a rigid bed. When using slopes, a control is necessary in the lock to keep the ships in the middle of the chamber and prevent them from running on the slopes. Such a type of construction is suitable only where there is a large surplus of water available. Ordinarily the slopes can scarcely be made steeper than 1 : 1. Thus, for a 3 m. (10 ft.) lock depth, a 10 m. (33 ft.) change in water surface and a 12 m. (39 ft.) width, a cross-section area of 280 sq. m. (3,000 sq. ft.) instead of one of 120 sq. m. (13,000 sq. ft.) must be filled. These figures show that such chambers generally are practicable in river locks with a rather small

lift, but not in canal locks. River locks also require the installation of perpendicular walls if power plants are combined with the layout.

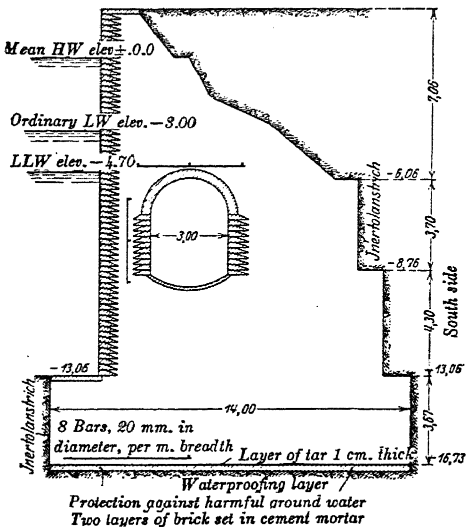
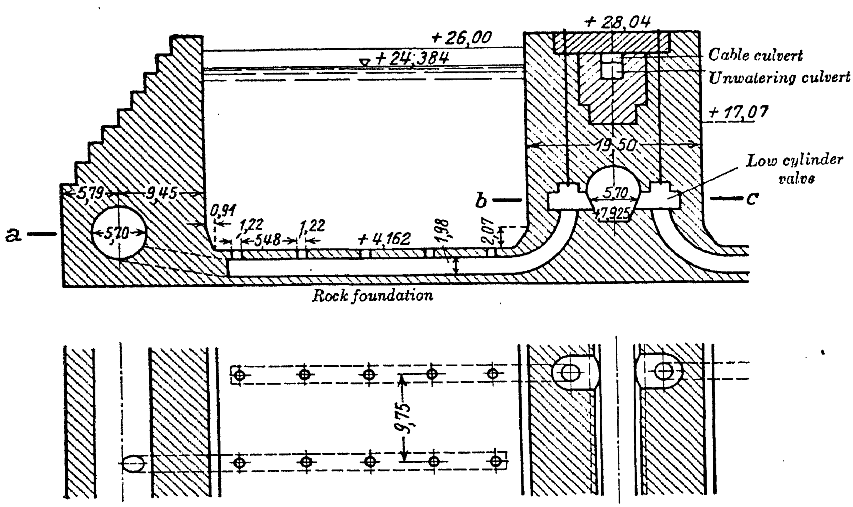


Fig. 365. Sea lock at Emden. Cross-section through wall of lock chamber

In the case of rivers a bulwark type of wall is often suitable. This style of structure was particularly successful in a lock at Hemelingen, a town near Bremen, Germany. Construction is facilitated by placing a floor or the floor framework in such a manner that the sheet piling fixes the upper edge of the floor. The stress in the bulwarks is then considerably less than when the soil is rammed in place without providing a massive bearing. It is difficult to install culverts when bulwarks are employed. Therefore, it is frequently necessary to resort to gate sluices or to use Mohr's stilling basin. At Hemelingen it was found advantageous to face the sheet piling with concrete anchored to the sheet piling from the bed to a height a little above the mean LW. [See



Figs. 366 a and b. The cross-section through the Gatun locks

Fig. 367 in which the 5 m. (16 ft.) pole serves as a scale.] Experiments indicate that this type of facing satisfactorily protects the iron against rust. It has even been found to free the sheet piling from rust, the latter being absorbed by the concrete and forming chemical combination therewith.[1]

The use of the bulwarks is limited by the lift of the lock. For a lock with 15 m. (49 ft.) lift and 3 m. (10 ft.) chamber depth, the sheet piling would have to be 19 m. (62 ft.) long from the upper edge of the lock platform to the bed. Whether a bulwark wall of this height is economical or even possible depends entirely upon the condition of the soil and the strength of the sheet piling. In general, bulwark walls appear to be limited to locks having a lift of not more than about 10 m. (33 ft.).

Although chamber walls must be designed particularly carefully, it is not justifiable to take abnormal, uneconomical precautions. The ground-water level back of the wall, for example, should not be assumed as high as the lock platform. Frequently the ground-water level is very low. At river locks the ground-water level in the vicinity of the upper end lies at about the same elevation as the headwater; at the lower end about the same as the tail-water. Accordingly, if the earth pressure and water pressure are to be treated separately, the wall need not be of uniform strength over its entire length; the strength may decrease toward the tail-water end. This consideration has not been given sufficient attention in the design of locks.

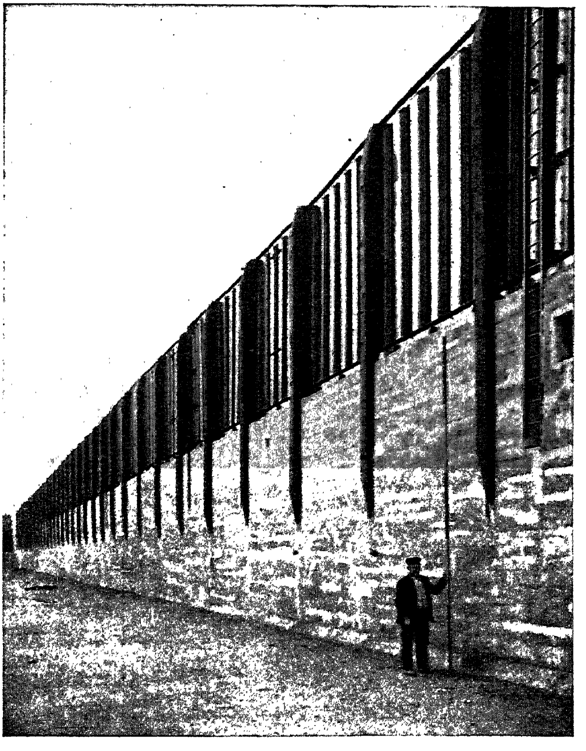


Fig. 367. Chamber walls of the lock at Hemelingen (Bremen). Sheet-pile wall faced with concrete

[1] Engineering report by Oberbaurat Kölle, Bremen.

A reduction in the cost of construction is possible by providing adequate drainage. The drainage system should be arranged so as not to

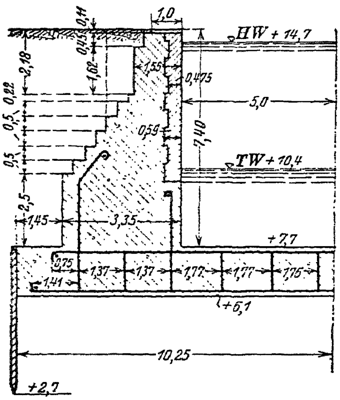


Fig. 368. Cross-section through chamber of barge train lock at Meppen. Dortmund-Ems Canal

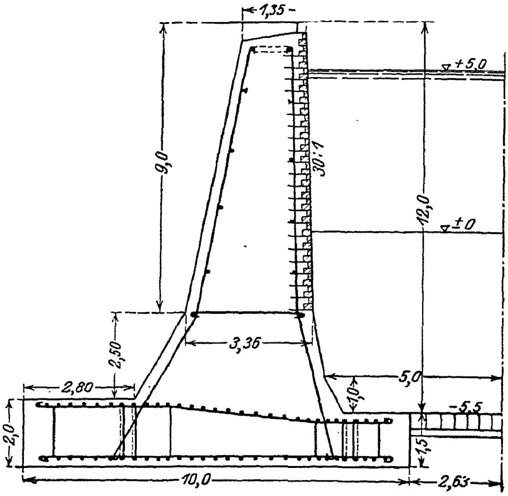


Fig. 369. Cross-section through lock chamber of the Rhine-Herne Canal

permit passage of the headwater behind the chamber wall when the upper gate is open. It should be possible to inspect the drainage system at any time. If the layout is not dependable at all times, it is worse than useless.

Walls are now built as multiple structures wherever possible, or at least where the construction cost is reduced by reinforcing. Particularly good designs of lock walls are given in Figs. 365 to 369. The one shown in Fig. 369 was built as an independent unit in view of possible settlement of the substrata due to mining operations. Fig. 370 presents a cross-section which has proven suitable and economical if there is a surplus of water available. It is used along the Oder and the Aller Rivers. The concrete mixes used are indicated in the sketch; the concrete block of 1:15 mix serves to overcome the earth pressure. However, the walls are said to have moved perceptibly. The upper space of 3.07 m. (10.1 ft.) width is partly waste space, resulting in a rather large water consumption. Fig. 371 shows the

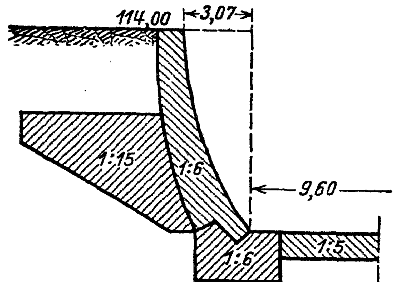
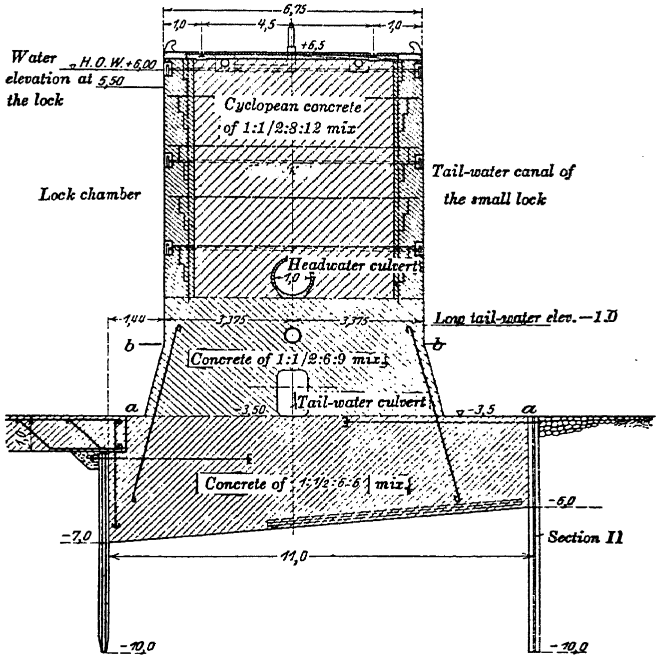


Fig. 370. Design of chamber on the Oder and Aller Rivers

wall between the Hemelingen barge-train lock and the tail-water canal of the simple lock lying beside it after restoration. During full train locking this wall serves as a weir toward the tail-water canal.[1]

A still more comprehensive design in which the entire section is constructed of reinforced concrete is shown in Fig. 372. This is a section of



H.O.W.=High head-water

Fig. 371. Center wall of lock at Hemelingen

a lock in the Tobol and the Kama River region designed by von Emperger. It is doubtful whether such a design is recommendable.

Lock walls in Germany are usually faced, although constructed of high-grade concrete. Even the best facing may freeze if it is not properly applied. A method of application proposed by Claussen, which was recently used in the lock at Anderten, is to be recommended (Fig. 373). The facing is constructed of dentated clinkers so that, for example, four layers of a half stone's thickness are uniformly followed by four layers of a full stone thickness. Dovetailed piers broadened in the back provide further anchorage with the concrete. These piers are stiffened by the insertion of round bars placed vertically. This type of facing wall has

[1] O. Franzius, Erfahrungen mit Gussbeton, *Beton und Eisen*, Vol. 3, 1914.

proven excellent in cohering with the concrete, whereas most of the other kinds of facings have occasionally become frozen and spalled off of the supporting wall. Fig. 374 shows the kind of facing used successfully by Zander in the Emden lock.

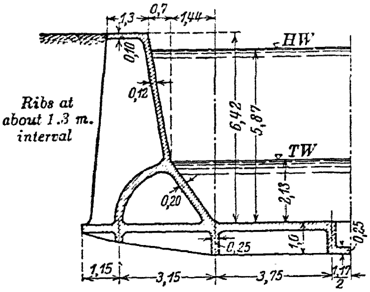


Fig. 372. Cross-section through lock chamber constructed of reinforced concrete. (Tobol and Kama River district)

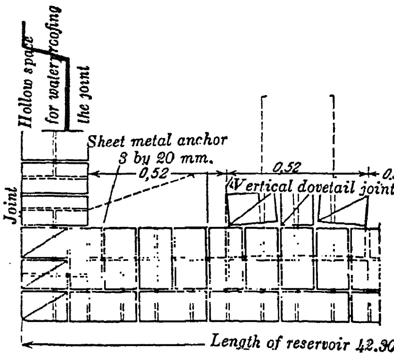


Fig. 373. Facing for the Anderten lock

In southern Germany concrete walls are frequently not faced, although the climatic conditions there are not materially milder than in northern Germany. Newly invented waterproofing compounds for concrete, such as *Sika* and others, might make possible the construction of concrete facing which does not absorb water and, accordingly, does not freeze. However, this would make it necessary to erect the dense concrete facing simultaneously with the structural concrete in order to obtain good bond between the two materials. Considerable saving in cost would result.

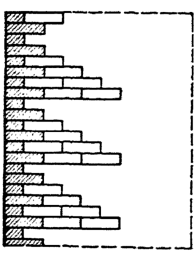


Fig. 374. Facing of the Emden sea lock with clinker of first and second quality

The floor design for the chamber may readily be computed if the latter acts as a gravity unit. Otherwise, a multiple type of reinforced construction may be used. Ordinarily a reinforced concrete floor in a lock consists of heavy transverse ribs (ribbed framework) with intermediate slabs. Floors consisting of rock fill are inappropriate because any projecting rocks might endanger the ships. A stone pavement should preferably be built upon a gravel base so that the current in the lock will not scour the ground below the stones. In large ocean locks having clay substrata, both the solid floor and stone paving have been omitted. The Kaiser lock in Bremerhaven, for example, consists only of massive bays and a chamber enclosed by walls.

G. LOCK GATES

a. General

Lock gates may be divided into the following classes:

1. Mitering gates and fan gates.
2. Tumble gates.
   α. With horizontal axis.
   β. With vertical axis.
3. Segment gates, lift gates, cylinder gates.
4. Pontoons.
   α. Floating pontoons.
   β. Sliding pontoons and pontoons on rollers.
5. Special types.

The present-day construction material for gates is primarily steel.[1] Wooden gates are now very seldom built. A combination of wood and steel, which occasionally has been recommended, has not proven satisfactory.

The durability of iron in water has been the subject of important investigations by Dr. Ederhof-Siegen. The studies were concerned primarily with iron in ocean water, but permit drawing conclusions regarding the behavior of iron in fresh water. The more important results are the following:

The deposition of silt causes special dangers to iron structures. It has been observed that iron parts upon which silt lodges rust more rapidly than other parts. Therefore, lock gates along the sea should be so designed that the projecting sheet iron edges are inside of the gate instead of outside, and so that all outside rivets are countersunk (Fig. 375). In Holtenau it was found that a layer of rust retards rusting if no strong current is present. New oxygen is required to continue the formation of rust but if oxygen is kept away or if its admission is retarded, as apparently is the case with a rust layer, rusting will be prevented or delayed.

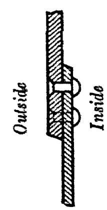


Fig. 375. Rivet joints for gates in sea water.

b. Mitering Gates

1. Strain of Walls, Inclination of Sill, Etc.

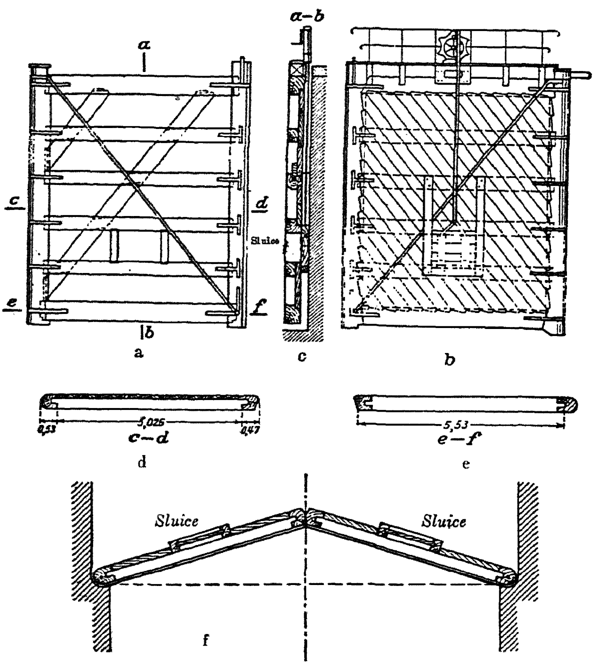
The mitering gate is to this day the most important type of gate. Diagrams of a typical wooden mitering gate, the Papenburger lock, are shown (Figs. 376 a to f) to illustrate the nature of such gates, but no detailed discussion concerning them will be given at this time. The

[1] High-grade wrought iron is now also considered as steel.

general designations have been retained to date from the time of the timber structures. The gate sheathing may be single or double; only single sheathing is necessary for small gates.

The use of a continuous quoin post, which revolves in a recess prepared for it, is now generally abandoned. Quoin posts are now replaced by individual mitering bodies for which an accurate preparation of the quoin recess is necessary. The mitering bodies, just as in the case of quoin posts, are supported eccentrically so that immediately after the gate begins to revolve it turns away from the quoin recess. Fig. 377 shows the arrangement of such a recess.

Assume the gate to form the angle φ with the normal to the lock axis; the clearances of the gate on all sides are s, s_1, and s_2. The angle through which the gate turns from a closed to an open position is $\varphi_1 = 90 - \varphi$. The quoin post is rounded cylindrically; its center when the gate is closed is C_1; when open, C_2. Accordingly, the center is displaced the distance $C_1C_2 = e$ (eccentricity) from the point of contact. The turning point M around which the quoin post turns to attain this motion is to be determined. M lies at the point of intersection of the perpendicular bisector of angle C_2C_1C, which is formed by the axis of the gate in the open and closed positions respectively. This can be readily proven geometrically.



Figs. 376 a–f. Wooden mitering gates in the Papenburg lock
a Front elevation b Rear elevation c Vertical section
d to f Cross-sections

The eccentricity is usually chosen as $e = 2$ cm. (.8 in.). The clearances are dependent upon the type of gate. Generally $s = 5$ to 10 cm. (2 to 4 in.), $s_1 = 5$ to 6 cm. (2.0 to 2.4 in.), enough space being allowed so that the bolt heads of the gate have sufficient room; usually $s_2 = 10$ cm. (4 in.). The length of the gate is calculated, taking into account these clearances, the width of the lock, and the angle φ.

The inclination of the sill is dependent upon the normal pressure

which is allowable on the wall of the bay. Consideration of the lock gate itself is not very important, since the gate may readily be designed to absorb any load to which it might be subjected. The strain upon the walls should receive first consideration.

When the gates are closed, the two leaves form a three-hinged arch with symmetrical loading (Fig. 378). The loading can readily be calculated from the difference in head on the two sides. The entire gate leaf, inclusive of its extremity in the recess, must be considered in computing the forces. Let the resultant of all

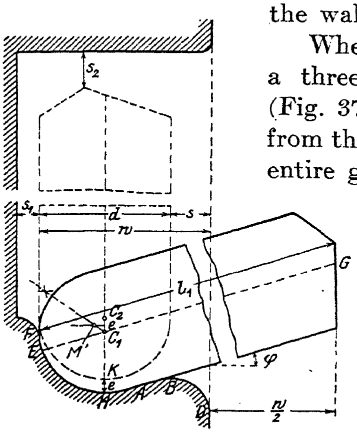


Fig. 377. Plan view showing closed and open positions of gate leaf

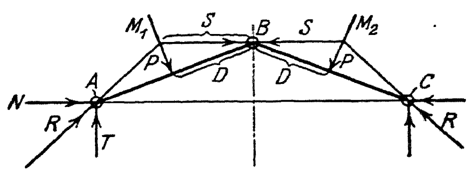


Fig. 378. Resolution of forces acting upon a mitering gate while closed

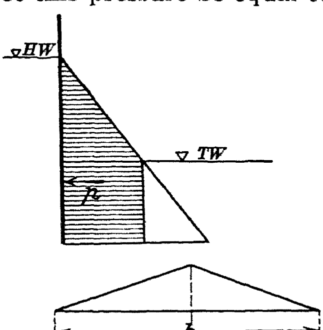
hydrostatic pressures for one wing be P. By using special pressure devices in contact with both gates and in the quoin recess, the hinged points are precisely determined; they are A, B, and C. The line of the resultant water pressure P is made to intersect with the normal to the lock axis through the middle hinged point B (point of intersection M_1 and M_2); thus giving the directions of forces R and S. The first is the pressure in the quoin post recess. To determine the effect of this pressure upon the wall, R is resolved perpendicularly and tangentially to the wall into the forces N (perpendicular to the wall) and T (tangential).

Force P must be expressed as the pressure of the lock gates acting upon a vertical strip 1 m. (3.28 ft.) wide. Let this pressure be equal to p (Fig. 379). Assume the width of the lock between the bearing surfaces of the quoin posts to be b_1. Then $P = \dfrac{b_1}{2 \cdot \cos \varphi} p$. From the force diagram (Fig. 380) it follows that

$$1.\quad R = S = \frac{P}{2 \sin \varphi} = \frac{1}{2 \sin \varphi \cos \varphi} \cdot \frac{b_1 p}{2} = \frac{b_1 p}{2 \sin 2\varphi}$$

$$2.\quad N = R \cos 2\varphi = .5\, b_1 p \cot 2\varphi$$

$$3.\quad T = R \sin 2\varphi = .5\, b_1 \cdot p.$$



Figs. 379 a and b. Water pressure upon a lock gate

A position of the sill should preferably be chosen so that the force R will resolve into forces N and T of about equal magnitude. These forces become equal when $\varphi = \frac{45°}{2}$; then $\cot 2\varphi = 1$ and $N = T = .5b_1p$. The force R then acts at an angle of 45° to the axis of the lock. This position of the sill ($\varphi = 22\frac{1}{2}°$) is readily fixed and is to be especially recommended.

The sill positions in use vary between the inclinations 1:5 and 1:2. In Germany the inclination 1:3 has been universally selected. The inclination of the sill of the Panama Canal lock because of the great pressures was made 1:2. A sill of 1:3 inclination forms an angle $\varphi = 18.5°$, that of 1:2 an angle $\varphi = 26.5°$. The mean of both angles is $.5\ (18.5 + 26.5) = \frac{45°}{2}$, which is the angle previously recommended. The inclination of the sill when $\varphi = 22.5°$ is 1:2.41.

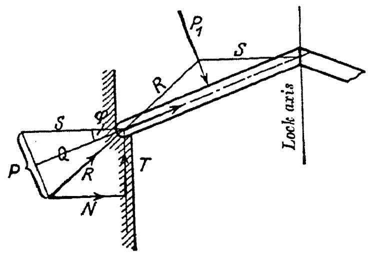


Fig. 380. Resolution of forces acting upon a mitering gate while closed

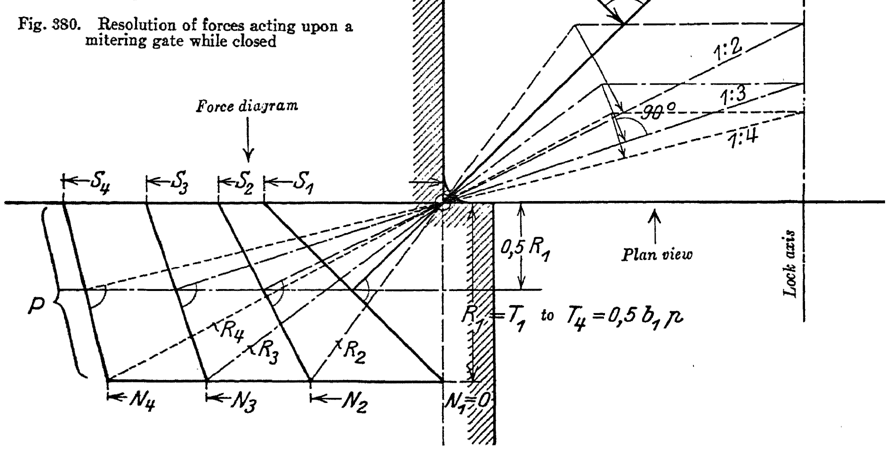


Fig. 381. Forces for various sill inclinations

Fig. 381 shows the position of R for various sill inclinations. Because of the great pressure on the side walls, position 1:5 must be eliminated as far as large locks are concerned. The variation in position and magnitude of R for inclina-

tions from 1:4 to 1:1 are shown. Since force T remains unchanged ($T = .5\, b_1 p$), and since the intersection of the horizontal through the end of T with the direction of R defines the magnitude of the latter, the position for R on the horizontal is at a distance of $T = .5\, b_1 p$. The figure indicates how rapidly the magnitude of N changes as the inclination of the sill with lock axis changes; for a sill inclination 1:4, N is nearly twice as great as T.

The wall of the bay must be calculated for stability under the influence of force N. The force must be distributed along a rather great length of the bay, but the stresses resulting because of this concentrated force must be analyzed and the walls must be reinforced accordingly. Force R acts at about the lower one-third point of the depth of the water, since the counterpressure of tail-water is very slight. The location of the line of action of the force should be accurately calculated.

2. Stressed Condition of Gate Leaves While Mitering

The magnitude of force R determines that of force Q normal to the gate. The force Q per meter depth is dependent upon the difference in head on the two sides of the gate, and below the tail-water level remains

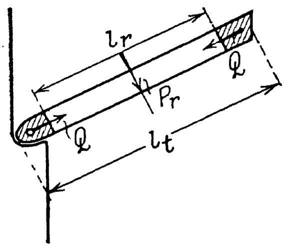


Fig. 382

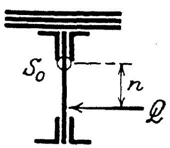


Fig. 383

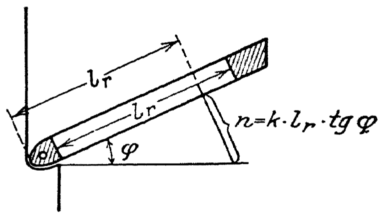


Fig. 384

Figs. 382–384. Stressed condition of the gate leaf while mitering (in Fig. 383 use η instead of n and in Fig. 384 $k{\cdot}l_r$ instead of $l{\cdot}r$)

constant for any depth. If force R is determined for a segment of fixed depth (Fig. 380), Q may be determined from the value of R. The magnitude of Q may also be found directly from force P, since $Q = \frac{P}{2} \cot \varphi$.

The analysis will be based upon the design of horizontal-girder gates because these have in general proved to be the safest type. Steel gates consist of framed sections and horizontal girders, which form the skeleton of the gate. The resisting moment of the vertical frame is so great that the stress in the girders due to bending need be computed only for the actual length, *i.e.*, from one inner edge of the frame to another (Fig. 382). If the length of the gate leaf is l_t, then the smaller girder length is l_r. Consider the cross-section of the gate shown in Fig. 383. Because of

flexure, extra material is required in compression. The stress in the middle of the girder, having a cross-sectional area F and resisting moment W, is

$$\sigma = \sigma_1 \pm \sigma_2 = \frac{Q}{F} \pm \left(\frac{P_r \cdot l_r}{8W} - \frac{Q\eta}{W}\right).$$

Because of the high position of the neutral axis (at the center of gravity S_0) and because of the position of the line of pressure of Q, which is intentionally moved downward, Q acts as a normal force causing a counter-moment $Q \cdot \eta$ to offset the bending moment resulting from the water pressure. The greater η becomes, the smaller the moment in the parenthesis becomes until limit 0 is reached. (Subsequent investigations in this chapter have as their purpose to determine whether the limit 0 can be actually attained.) In the following P_r and Q are expressed in terms of p.

$$P_r = p \cdot l_r \text{ (for the length of the girder),}$$

and

$$Q = \frac{P}{2 \tan \varphi} = \frac{p \cdot l_t}{2 \tan \varphi} \text{ (for the gate length)}$$

Therefore

$$\sigma = \frac{p \cdot l_t}{2 \tan \varphi F} \pm \left(\frac{p \cdot l_r^2}{8W} - \frac{p \cdot l_t \eta}{2 \tan \varphi W}\right) = \frac{p \cdot l_t}{2 \tan \varphi}\left[\frac{1}{F} \pm \frac{1}{W}\left(\frac{l_r}{l_t} \cdot \frac{l_r \tan \varphi}{4} - \eta\right)\right].$$

If l_r/l_t is designated by k, and $l_r \tan \varphi$ by n, then in present-day designs k is about .95 and n the value indicated in Fig. 384. Lay off a distance $k \cdot l_r$ along the sill line from the quoin recess, and at the extremity erect a normal to the sill; the intersection of this normal with a line joining the quoin post recesses cuts a length equal to $n\ l_r/l_t \cdot l_r \tan \varphi$. Hence,

$$\sigma = \frac{pl_t}{2 \tan \varphi}\left[\frac{1}{F} \pm \frac{1}{W}(.25\ n - \eta)\right].$$

Thus the bending moment becomes 0, if the distance between the line of action of the miter pressure and the neutral axis is $\eta = .25n$. For a lock 12 m. (39 ft.) wide having sill inclination of 22.5°, $n = 2.4$ m. approximately, and $n/4 = .6$ m. In this case it would be possible but probably uneconomical to make the gate thick enough to produce zero bending moment. The disproportion becomes even greater in ocean locks. A certain amount of bending moment is therefore acceptable but it should be kept small by increasing η.

Formerly it was attempted to exclude tension stresses from the gates altogether by designing cylindrical-shaped gate leaves. However, statical advantages are better obtained by parabolic-shaped gates. In con-

struction, more emphasis is given to obtaining the simplest forms possible, so that fabrication is not complicated by work of bending steel into special forms. The leaves of large gates should preferably not be curved in plan, but constructed with broken outer lines according to the pattern of the mitering gates in the Kaiser Wilhelm Canal. Nevertheless, it is of interest that to the present time gate leaves are frequently constructed with a curved outer sheathing. (This is true of the gates at Hemelingen, Doerverden, and of the Panama Canal.) Hence the increase in expense due to bending the steel does not seem to be as great as is often maintained.

3. Action of Mitering Gates During Suspension; Insertion of Air Chambers

In the suspended position the mitering gate is subject to forces similar to those of any other gate. It is assumed that air chambers increasing the buoyancy are built into the gate. The vertical force exerted upon the lower pivot then is $S=G-A$, the horizontal force which acts as a tension force at the top and as compression force at the bottom is equal to

$$Z=\frac{G\cdot g-A\cdot a}{h}.$$

The direction and magnitude of the forces may be determined graphically just as readily as algebraically (Fig. 385). The pull of the suspended gate must be considered when computing the stresses and stability of the wall. This pull may be distributed along a rather great length of the wall. Of course, proper design measures (reinforcing, etc.) are necessary under certain circumstances. It is advisable if possible to neglect the effect of the air chambers when designing the walls, because the chambers may become leaky[1] as a result of impact of ships against the gates.

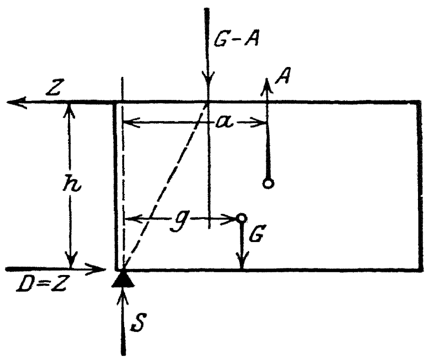


Fig. 385. Forces encountered in connection with a hanging gate

4. Heights of Gates; Protective Measures Against Tidal Floods

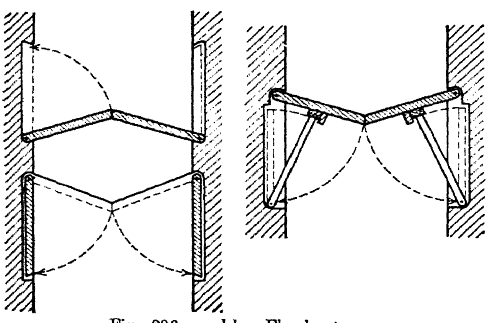
For inland lock gates about 20 cm. (8 in.) height is allowed above the highest water level to which they are subjected. The gates of locks

[1] This has happened rather frequently especially in the case of the locks in Hemelingen (Bremen).

on great inland lakes, such as the Great Lakes of North America, form an exception because the occurrence of high waves necessitates precautions similar to those necessary along the ocean front. At times when navigation in rivers ceases in consequence of the stage exceeding a fixed, highest navigable water level, the water is permitted to flow over both the lock platform and gates. For canalized rivers, therefore, the upper edge of the gate is extended only 20 cm. (8 in.) above the highest navigable water level. The gates extend somewhat above the highest navigable water level so that water will not flow over the gate continuously or in waves at this stage.

On the ocean front and on great inland lakes, the upper edge of the gate must protrude .6 to 1 m. (1.9 to 3.28 ft.) above HHW depending upon the height of the waves. The elevation of the top of the gate enters into consideration particularly in the case of tidal-wave gates which form a special part of sea locks. The tidal gates, together with the outer bay, form a continuation of the sea dike; they are subject to dike inspection and in Germany are under the care of the dike association.

Figs. 386 a and b contain two types of construction of tidal gates. The customary construction of special tidal gates in Germany is shown in Fig. 386 a. They lie on the ocean side of the lock so that the entire inner part of the lock, inclusive of the outer-harbor gates, is protected by them. Fig. 386 b shows a French arrangement with countergates. The countergates consist simply of frames which serve to keep the lock gate braced against the excess pressure of the outer water. The French arrangement is considerably cheaper than the German, but it does not offer the same degree of safety. The great jolt caused by the onrushing waves might lead to a loosening of the frames; furthermore, the collar and foot bearings are stressed in the opposite direction from that which ordinarily obtains. These bearings during normal mitering carry no load whatever, since the entire force is transferred directly to the masonry. In the countergate design, the bearings of the main gate are required to take part of the force of heavy wave thrusts. This arrangement, therefore, is not to be recommended. In view of the importance of an adequate dike to the protection of lowlands, safety should not be reduced by economizing on the cost of tidal gates.



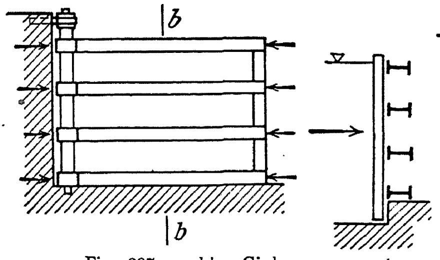
Figs. 386 a and b. Flood gates
a. German design b. French design

The sill should project as little above the floor as possible because a high sill weakens the floor. In ordinary canal locks a sill height of .3 m.

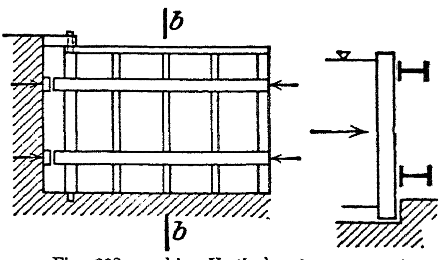
(.9 ft.) will suffice; but for ocean locks, the height must be as much as .5 m. (1.6 ft.) or more.

5. Post Gates or Girder Gates; Construction Procedure

In case of very long gates which are not very high, it is not suitable merely to stiffen them with horizontal girders. It is frequently desirable to place a horizontal girder at about the height of the headwater, and vertical stiffening posts at suitable intervals. This type of construction allows much greater pressures to be transferred to the sill than would be possible by a supporting frame purely of horizontal girders. Allowing a large amount of the gate pressure to be carried by the sill tends to break the sill, and the saving in gate cost compared to the cost of the entire



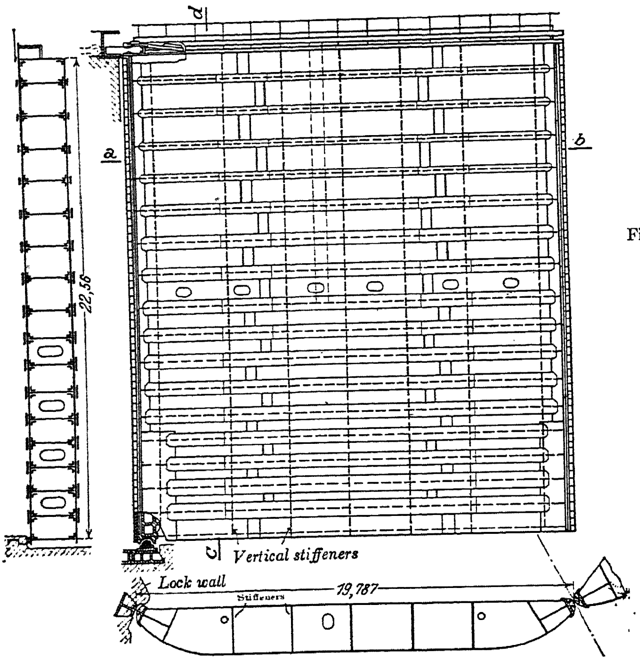
Figs. 387 a and b. Girder arrangement.



Figs. 388 a and b. Vertical post arrangement

lock structure does not justify the possibility of the entire installation becoming disabled. If uprights are to be used in the gate, the design should be such that the lower ends of the vertical members are supported at a short distance above the port sill by a lower girder, which together with the upper girder forms the bearing surface for the uprights (Fig. 388). By the use of large, well-anchored bearing plates, the lower girder pressure, which may become very great, may be counteracted by the wall because the resisting force of the entire lateral wall is available. Such a bearing point, however, must be firmly anchored by heavy steel reinforcing.

General specifications governing the design of steel structures are applicable to lock gates. But since the latter lie alternately under water and in the air, utmost simplicity in construction should be used. Web plates are to be preferred to latticework. The real support of gates consists of the quoin post (along the wall) and the miter post (at the end of the gate) which, in case of rather large gates, are made of plate or box girders. The horizontal girders join these end posts. The horizontal members should preferably be plate girders but may be lattice girders. The metal sheathing should preferably be placed directly upon the flange angles and the cover plates riveted over the sheathing. In this



Figs. 389 a–c. Mitering gates of the Gatun lock, showing section, elevation, and plan respectively

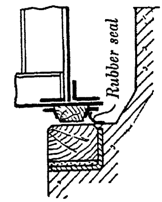


Fig. 390. Waterproofing at the sill of a lock

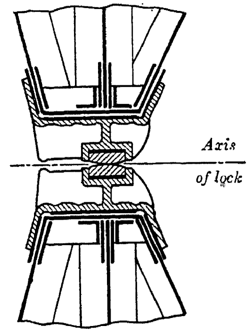


Fig. 391 a. Miter post joint

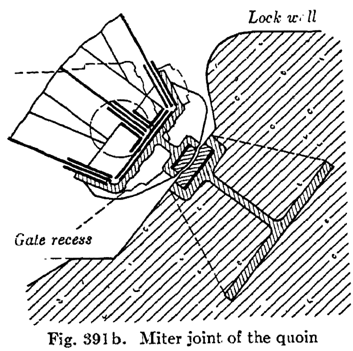
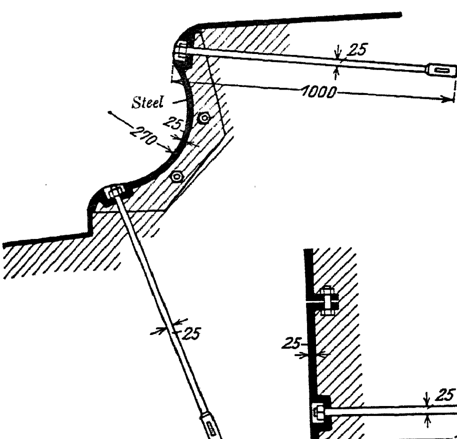
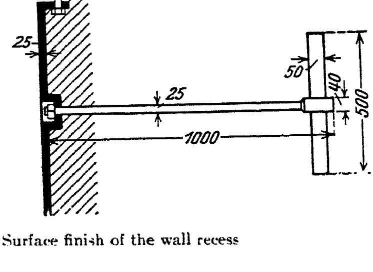


Fig. 391 b. Miter joint of the quoin





Figs. 392 a and b. Surface finish of the wall recess

way all of the riveting and fitting becomes cheaper and maintenance less complicated. Since the cover plates differ in length, corresponding to the magnitude of the moments, the metal sheathing would have to be bent at several points to make it rest upon the laminated cover plates. Putting the plates for the girder flange on top eliminates this work and makes possible a more impervious structure. This arrangement was employed on the Gatun locks (Figs. 389 a to c). Extra cover plates are not necessary in case of small gates; for these, standard sections frequently suffice. The continuous plate covering riveted on these channel-beams or I-beams functions as a unilateral reinforcement, and may to a certain extent be computed as such. The horizontal girders must be connected by intermediate vertical uprights. The horizontal girders are continuous through the entire width of the gate because they are designed to carry the greatest load; the uprights may extend in sections from girder to girder. If the girders are wide, the uprights may pierce them. Curved structures, for reasons already mentioned, should be avoided if possible. It is noteworthy that in recent times many gates with a curved cover have been constructed. The gates in the Gatun locks in the Panama Canal form a large scale example (Fig. 389). Below the tail-water level, the girders are uniformly spaced, but from this elevation upward the spacing increases. Frequently in theory no girder at all is required at the top; but an upper frame must be added to provide local stability. The service walkway is supported by the top frame. Local stability is of vital importance in gates; general stability of the whole structure alone does not suffice. Careful consideration should be given to evaluating the forces to which gates may be exposed as a result of ship thrusts, ice pressure, beating of the waves, and the like. Consequently, many parts must be built much stronger than would otherwise be necessary.

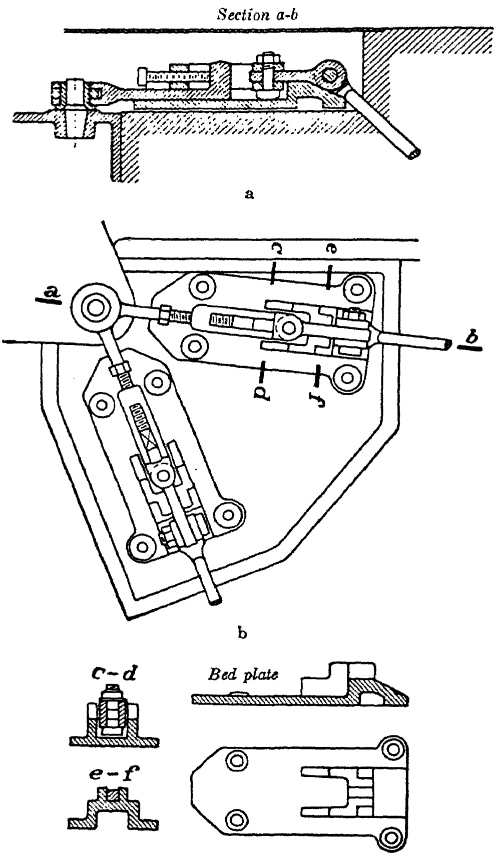
If air chambers are to be used, they should preferably be put under air pressure so that a leak might be immediately detected by the escape of compressed air. A very efficacious arrangement was effected in the old locks of the Kaiser-Wilhelm Canal by introducing compressed air through hollow upper gudgeons. If the gates are large, metal sheathing is usually put on both sides. This provides additional security, and air chambers are formed naturally if the riveting is water-tight. In such instances care must be exercised to provide adequate water chambers so that the air chambers do not cause an undesired pressure of the gate against the upper quoin-post bearing.

In some cases the miter-post bearing strips extend around the end of the gate to the tail-water side. The strip is then preferably placed in front of the miter post along the tail-water side. For best results, the

miter post should be constructed as a continuous steel beam furnished with a special bearing face. The only material to be considered for the bearing surface is hard wood, such as oak or good beech wood, protected against decay by impregnation. It must be borne in mind that wood under water has but 50 per cent of the strength of wood above water.[1] Destruction at locks in consequence of crushing the miter-post bearing strips and breaking the gates is traceable to a decrease in the strength of wood below water.

6. Various Types of Bearings; Anchorage

The diagrams in Figs. 391, 394 and 397 to 399 are given as examples of bearings, respectively, at the quoin-post and miter-post ends of the horizontal girders used in the framework of mitering gates. The figures are self-explanatory. The bearings of the gates for the Gatun locks (Figs. 391 a and b) were particularly carefully designed. The material used in large gates is cast steel; that in smaller ones, cast iron. Forgeable castings are now considered favorable.



Figs. 393 a–c. Upper collar bearing in plan and cross-section

The quoin-post recess must be lined with adequate bearing material firmly anchored in place (Fig. 392). Eccentric bearings are of the same type as in the old wooden gates. Thus, immediately after the gate begins to revolve from a closed position, there is a marked displacement of the two miter posts away from each other. Frequently the entire quoin-post recess is lined with a casting. It is advis-

[1] Lang, *Das Holz als Baustoff*, Wiesbaden, Kreidels Press, 1915.

able to finish and completely assemble the gate and recess lining in the factory. If they cannot be shipped assembled, they should be delivered to the building site in sections. Construction at the lock site is practicable only in case of gigantic gates. At the top of the quoin post is a machined gudgeon which is held by a collar bearing. This collar is anchored into the wall in two directions. The upper gudgeon may be either in single shear or double shear. Figs. 393 a to c show the first arrangement; Figs. 394 a and b the second. The single shear design comes into consideration in rather small lock gates, the double in large gates. The single shear type requires a gudgeon thickness of

$$d_1=\sqrt[3]{\frac{10}{K_b}}\sqrt[3]{z\cdot l},$$

in which z is the pulling force on the upper edge of the gudgeon, l the gudgeon length, and K_b the allowable stress in the material. The resisting moment of the circular area is $W=0.1\ d^3$.[1]

For the double shear,

$$d_2=\sqrt[3]{\frac{2.5}{K_b}}\sqrt[3]{z\cdot l},$$

in which z acts at the center of the gudgeon. As an example, for $K_b=$ 1000,

$$d_1=0.215\sqrt[3]{zl}\quad d_2=0.136\sqrt[3]{zl}.$$

The allowable stress must be kept low because of the wear and tear.

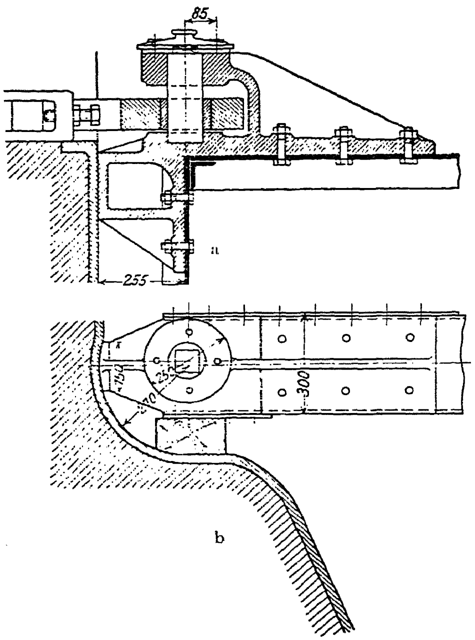
Recent designs in which ball bearings are put into the collar bearing introduce refinement that seems superfluous because of the slow movement of the gates, the slight expenditure of power, and the insignificance of the journal friction as compared to other retarding forces. The crude gate-hinge bearing should not be unduly complicated. In general, it is recommended to design the gudgeon and then add several centimeters thickness for wear and tear.

The upper gudgeon is a rigid constituent part of the revolving column. However, the gudgeon collar must be designed to permit displacement in all directions while being installed and also later if necessary. Although adjustable, it must be rigidly fastened with anchors, screws, or wedges so that there is no movement of the ring during operation of the gate.

Figs. 393 to 395 indicate various designs. The older designs in which the final setting is fixed during construction by the insertion of wedges, are no longer recommended. It is preferable to fix all movable parts

[1] More accurately $0.0982\ d^3$.

with screws. The arms of the yoke are adequately spread so that they point practically in the direction of the tension forces in the end positions of travel of the freely suspended gate. This condition is not wholly attainable if the gate is to hang wholly within the recess, because the anchoring would be too close to the water side of the wall. In order to assure a completely fixed position of the gudgeon collar, it is advisable to supply a suitably shaped cast-steel casing for lining the recess for the gate; the anchorage for the collar is thereby made more secure.



Figs. 394 a and b. Collar bearing and gate stop of the Scheitniger lock at Breslau

The anchors must be built into the wall at the time it is constructed. Frequently the concrete is not poured until the gates have been placed, although it is preferable to complete this operation before setting the gates. In many instances the anchors are permitted to extend from the wall in links to which the fastening bolts are linked, thereby introducing a hinge between the collar and anchor. The anchors should preferably extend to the rear edge of the wall. If no cavities interfere, the anchors may be inserted rectilinearly. When shafts or the like interfere, the anchors are placed around them. Pressure bearings are inserted in the masonry at bends in the anchorages.

The lower bearing of most gates, even in more recent times, has been constructed immovable. Figs. 396 and 397 show designs which are no longer to be recommended. A pintle turned upward rests in a bearing plate; a concave bearing plate fastened to the bottom of the quoin post turns upon the pivot. By reversing the old pivot bearing, sand is prevented from getting between the pintle and the disk in large quantities. Ordinarily the pintle is sufficiently thick if the resultant of the forces acting upon it falls within a radius, relative to the axis of the pivot, which is not greater than one-third the radius of the outer edge of the pivot.

The radius R is dependent upon force P and the material used. According to Landsberg,

$$R \geqq 0.691 \sqrt{\frac{P}{K(1-\cos^3 \alpha)}}, \text{ in which } \tan \alpha \geqq \frac{Z}{V}.$$

P is stated in tons and K in tons per sq. m.; Z is the horizontal anchor tension, and V the weight of the gate after deducting buoyancy. The gudgeon should rest independently in a special bed plate to allow for

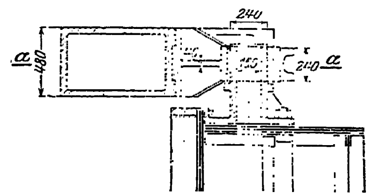


Fig. 395a

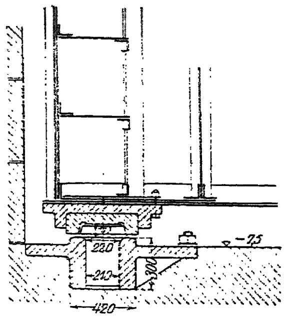


Fig. 396a

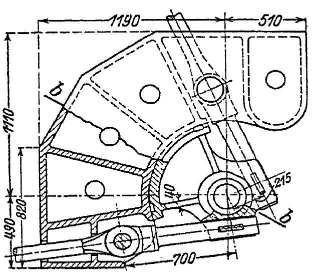


Fig. 395b

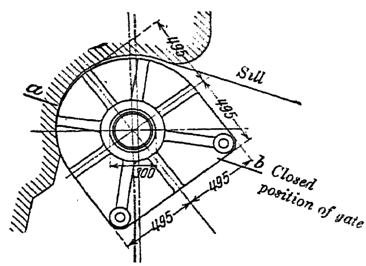


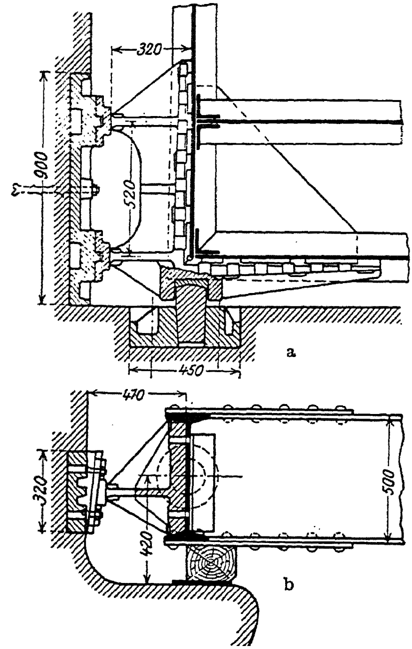
Fig. 396b

Figs. 395–396. Bremerhaven chamber lock. Collar and thrust bearing
Figs. 395 a and b. Collar bearing Fig. 396 a and b. Pintle. Old type of design

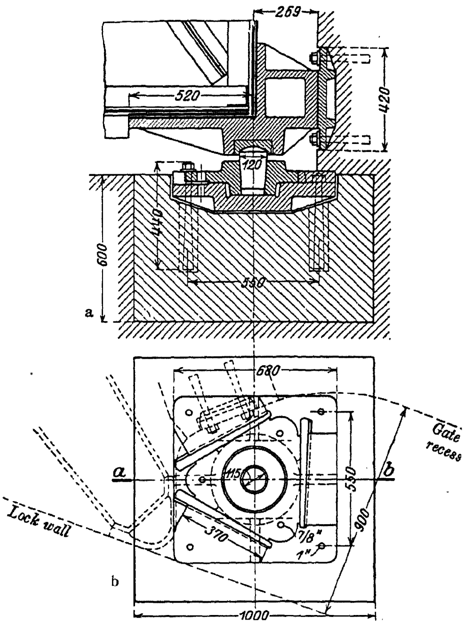
periodical renewal. The disk bed must also be exchangeable. Hotopp's design for the locks of the Elbe-Trave Canal (Figs. 398 a and b) introduced an improvement in the design of gudgeon bearings. Hotopp placed the supporting plate with the gudgeon upon a lower base plate over which the former may be moved. The upper bearing plate is triangular, and is maintained in position laterally by wedges. This arrangement makes possible a slight displacement in all horizontal directions, and exact placement is simplified. However, this method does not obviate the danger of rupture to the gudgeon when solid objects are jammed between the gate and stop bearing.

The danger of breaking the bearing caused by objects becoming jammed behind the quoin post is greatly minimized, if not entirely eliminated, by Buchholz' invention (Fig. 399). Buchholz developed a

supporting spindle in the form of a swinging support. This support accurately follows the pressures that are exerted on the gate. When the

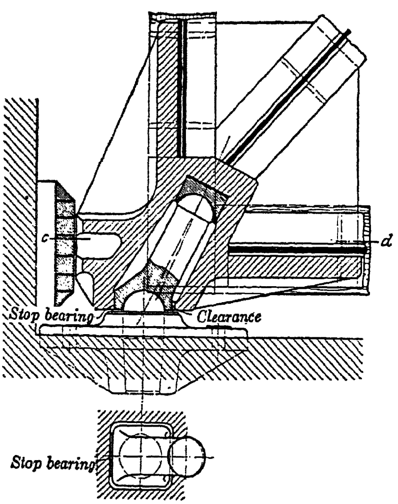


Figs. 397 a and b. Pintle of a mitering gate in the tail-bay of the lock at Troja (antiquated)
a Vertical section b Horizontal section and stop bearing



Figs. 398 a and b. Hotopp bearing

a Plan view of bed plate. The section through the pivot recess is shown by dotted lines



Figs. 399 a and b. Buchholz bearing for mitering gates

gate miters, it lies in the quoin recess; when the gate is suspended, it turns away from the recess. The swinging arm then lies against a flange so that the movement of the gate away from the recess ceases after turning through a small angle. Buchholz' design is now the best type of bearing. It overcomes most of the friction force ordinarily encountered in the quoin recess, because the quoin post turns in a free space. Furthermore, the danger of breaking the gudgeon bearing is very small with this type of bearing. Formerly fail-

ures in gudgeon bearings were not unusual. In choosing the type of metal to be used, it should be remembered that various metals must not rest on one another in ocean water because of the destructive influence of galvanic currents which would develop within a short time.

7. Operation Appliances for Mitering Gates

a. Resistance to Movement

The resistances to the movement of mitering gates are as follows:
1. Friction along the collar and pivot bearing.
2. Inertia resistance of the gate itself to acceleration.
3. Water pressure due to difference in water elevation on the two sides of the gate.
4. Resistance of the water to movement.
   a. In consequence of the movement against the water (corresponding to the towing resistance of ships).
   b. In consequence of the damming up of the water as the gate opens.
5. Resistance of cakes of ice, silt, sand, and the like.

The resistances enumerated in items 1 up to 4 theoretically may be determined approximately; those under 5, on the other hand, are not readily evaluated by theoretical considerations.

The force Z may be considered as resolved into Z_1 and Z_2 (Fig. 400). However, in the final analysis, force Z must act upon the circumference of the upper gudgeon as a total force. For simplification, Z may be considered as a single force acting at a point. This force then acts on the lever arm of $.5\ d_1$; the turning moment, assuming a friction resistance μ between collar and journal, is

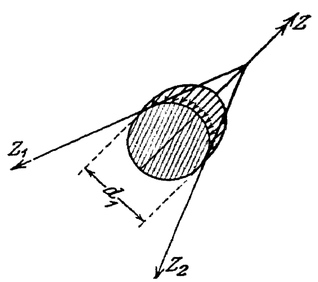


Fig. 400. Forces acting upon the collar bearing

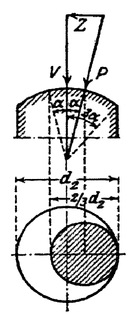


Fig. 401. Forces acting upon lower pintle.

$$M_1 = .5\mu \cdot d_1 \cdot Z,$$

and if

$$Z = V \cdot \tan\alpha, \quad M_1 = .5\mu d_1 V \cdot \tan\alpha.$$

The resistance of the lower pintle can be determined in a similar manner (Fig. 401). Then $P = V/\cos\alpha$. Force P may be considered to

act upon the pintle distributed over a circumferential surface of $2/3d$. The whole surface pressure may then be assumed as replaced by the pressure of a single force P, which lies at the distance $.25 \cdot 2/3d_2 = 1/6d_2$ from the center of rotation. If the same friction coefficient μ be assumed, then the turning moment is

$$M_2 = \frac{\mu \cdot d_2 \cdot V}{6 \cos \alpha}.$$

The total moment of the bearing friction then is

$$M_1 + M_2 = \left(.5 \cdot d_1 \cdot \tan \alpha + d_2 \frac{1}{6 \cos \alpha}\right) \mu V.$$

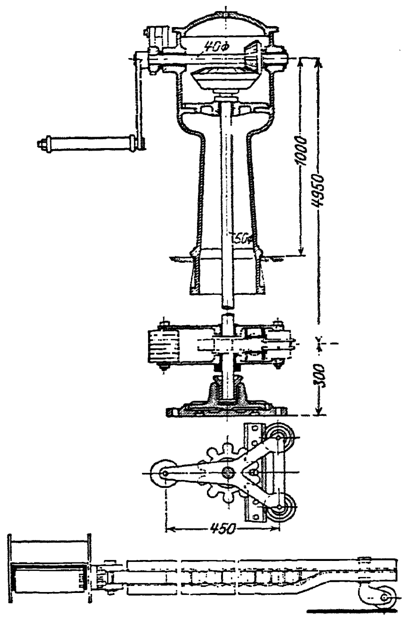
The other turning moments can be calculated in a similar manner. All of the factors which can be calculated in this way, however, are only a small fraction of the combined forces usually occurring. The resistances enumerated under number 5 particularly are based wholly upon arbitrary assumptions; furthermore, the resistance under 4*b* is very hard to determine. Being able to open the gate against excess water pressure has great influence upon normal operation [Δh being ordinarily 10 cm. (3.94 in.)].

If all these exact computations are made for the average size lock-gate, then for gates that have driving machines of 10 HP in reality, theoretically only a 5 to 6 HP motor is required or perhaps only one-half the capacity of one which has proven correct in practice. The resistances consisting of jams of the gate, resistance of silt and ice, and the like are difficult to formulate mathematically. The cost of a 15 HP machine compared to one of 10 HP is not so significant as to justify the risk of obtaining too small a motor. Moreover, the power consumption of these motors is slight. Nowadays the time required for opening the door seldom amounts to more than half a minute. More than 20 lockages per day seldom occur; that would be a period of operation of 40 minutes a day, assuming that one minute is required for opening the gate. Considering 300 locking days, a motor would operate 200 hours annually. For 5 HP excess 3,000 kilowatt hours would be the excess consumed for four gates. The cost of this amount of power would be comparatively insignificant; hence, it is immaterial whether 15 or 10 HP motors are installed.

β. The Various Kinds of Gate-Operating Installations for Mitering Gates

The oldest mode of gate propulsion is that of man power. In its simplest form, this consisted of opening and closing gates by means of

poles. Next, hand-operated windlasses were installed. A simple device of this kind consisting of a rack and pinion arrangement is shown in Figs. 402 a to d. The rack, which is fixed to the gate, is held against the cogwheel by the counterpressure of rollers. The counterrollers and the cogwheel must be placed between two metal disks which are free to revolve about some transverse axis, the axis of the cogwheel being most suitable. The rack is then free to change its direction; if the metal plates were fixed in position, the rack and pinion would soon become jammed.



Figs. 402 a to d. Gate drive by means of rack and pinion for hand operation of the lock at Brieg

A later development consisted of a chain drive in which the ends of the chains were fixed to the bottom of the gate in such a way that one chain could be used for locking, the other for opening the gate. This system of movement has been used in large ocean locks but now is entirely antiquated.

There are a great many varieties of operating devices for gates, and only those which may be considered among the best are described herein. All modern methods of operation are electrically equipped. Almost all

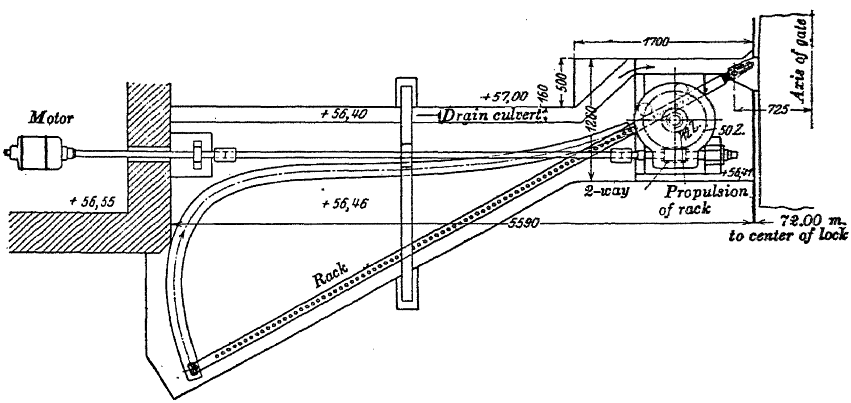


Fig. 403. Mitering-gate actuation by means of rack and pinion as proposed in a design by Freund-Starkehoffmann-Maschinen A.-G. of Berlin, for the lock at Münster

employ a connecting rod which is pressed forward and drawn back in some manner. This connecting rod, in consideration of the waves and currents encountered, must be elastically connected to the gate, *i.e.* by inserting a spring between it and the gate. Rigid connections are easily broken. Some of the better arrangements include the following:

1. A rack and pinion arrangement (Fig. 403).
2. An electrically operated travelling crab which runs on a fixed dentated connecting rod.
3. Segment of a toothed wheel which is operated back of the gate and moves the gate by means of a lever arm (Figs. 404 and 405).
4. Turning disk propulsion with connecting rod (Panama) (Figs 406 and 407).

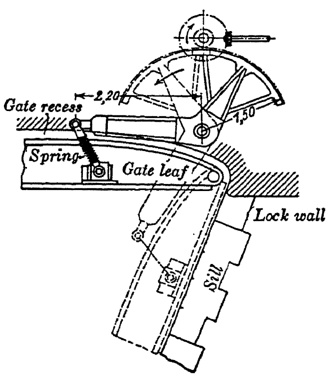


Fig. 404. Mitering-gate actuation at Niederfinow

5. The Freund-Starkehoffmann engine with connecting rod.
6. The Hotopp, Nyholm, and Franke types, respectively, and propulsion by propellers (Figs. 409 to 411).

The following statements are pertinent to the foregoing classifications:

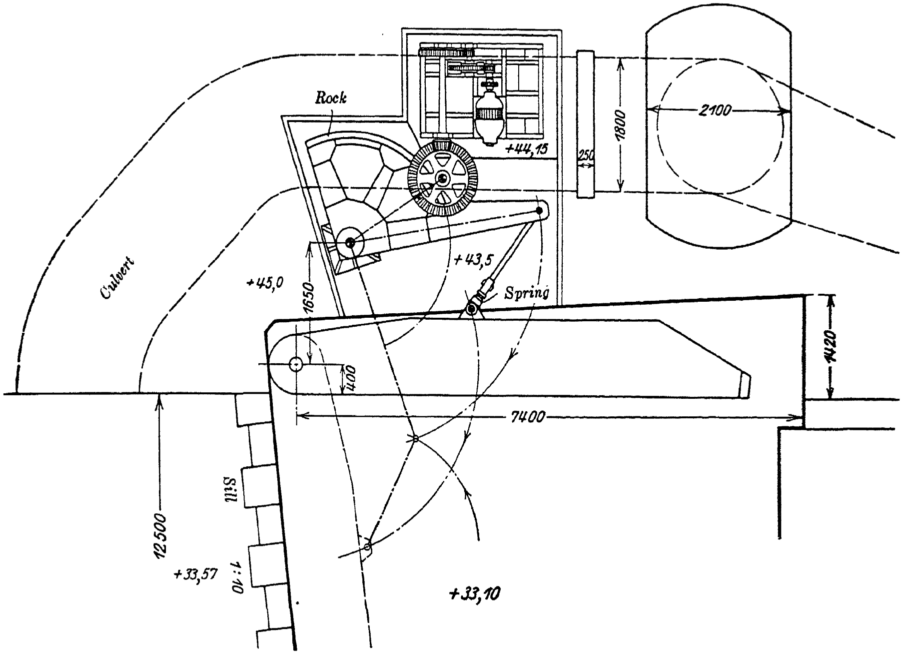
Concerning 1. (Propulsion by means of rack and pinion.) The connecting rod must be fastened to the gate elastically. The drive pinion must be placed as close as possible to the front edge of the recess, so that the connecting rod will be of minimum length. Since the rod is subject to high bending stresses, it must be designed for flexure. The path of the wall-side end of the connecting rod must be determined in order to fix the proper width of wall cavity.

Concerning 2. This mode of propulsion has been used successfully with tumble gates but may also be used on mitering gates. The direction of the pressure exerted on the gate may be arbitrarily fixed with this type of layout.

Concerning 3. The segment is supported independently of the gate. A strong buffer spring is fastened between the gate and the end of the arm which actuates the gate.

Concerning 4. The Panama arrangement rests on the basic thought that, in the two end positions of the gate, the connecting rod lies in such a position that its ends, that is, its points of contact with the gate and the disk, respectively, lie in a straight line through the center of rotation of the disk. If the disk rotates beyond either end position, the gate movement reverses, thereby obviating the possibility of the operating mechan-

ism or gate becoming jammed. The geometrical solution is contained in the taction problem of Apollonius Pergaeus, which may be found in textbooks on geometry. In a more recent solution by the *Freund-*



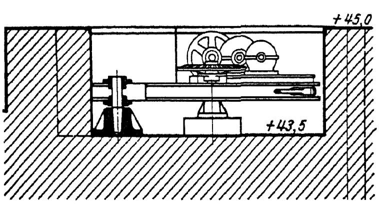
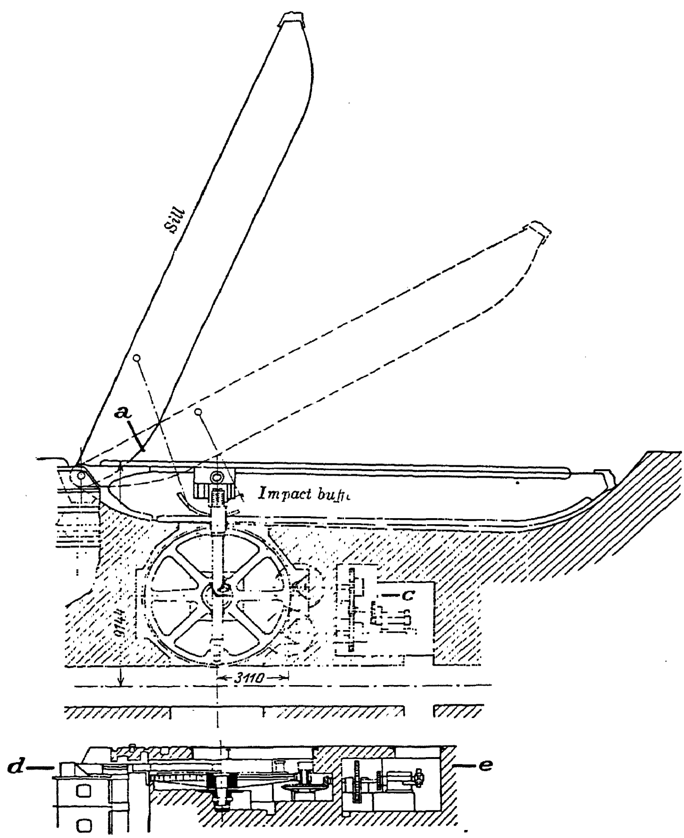


Fig. 405. Mitering-gate actuation with crank wheel sector. Designed by the *Freund-Starkehoffmann-Maschinen A.-G.* for the Weser descending lock at Minden. Cross-section and plan view

Starkehoffmann Maschinen A.–G. for the lock at Minden, the mathematically exact method was not used (Fig. 407). The design is presented for the purpose of comparison with other locks at Minden.

Concerning 5 (Fig. 408). The connecting rod is hinge-connected to the gate and to the wall, and is provided with an intermediate hinge. Propulsion takes place at the middle joint through a second connecting rod. This arrangement also has the advantage that no excess stress is brought to bear upon the gate.

Concerning 6. The modes of actuation named under 6, with the exception of the propeller, also employ the connecting rod. In all three, the head on the lock is used to generate pressure. The Hotopp type produces compressed air with the aid of water pressure in a very interesting manner, but is no longer practical for present-day designs. The compressed air is used to raise and lower a heavy diving bell which moves



Figs. 406 a and b. Gate drive of the Gatun lock on the Panama Canal

within a vertical shaft. Fig. 434 shows the generation works; Fig. 409, the method of actuating mitering gates. When the bell is filled with compressed air through the metal tube, it becomes lighter than its counterweight, and is raised, thereby actuating a pulley. If the com-

pressed air is released from the bell, the bell sinks. A connecting rod having one end fixed to the gate is moved by the pulley. Hotopp's arrange-

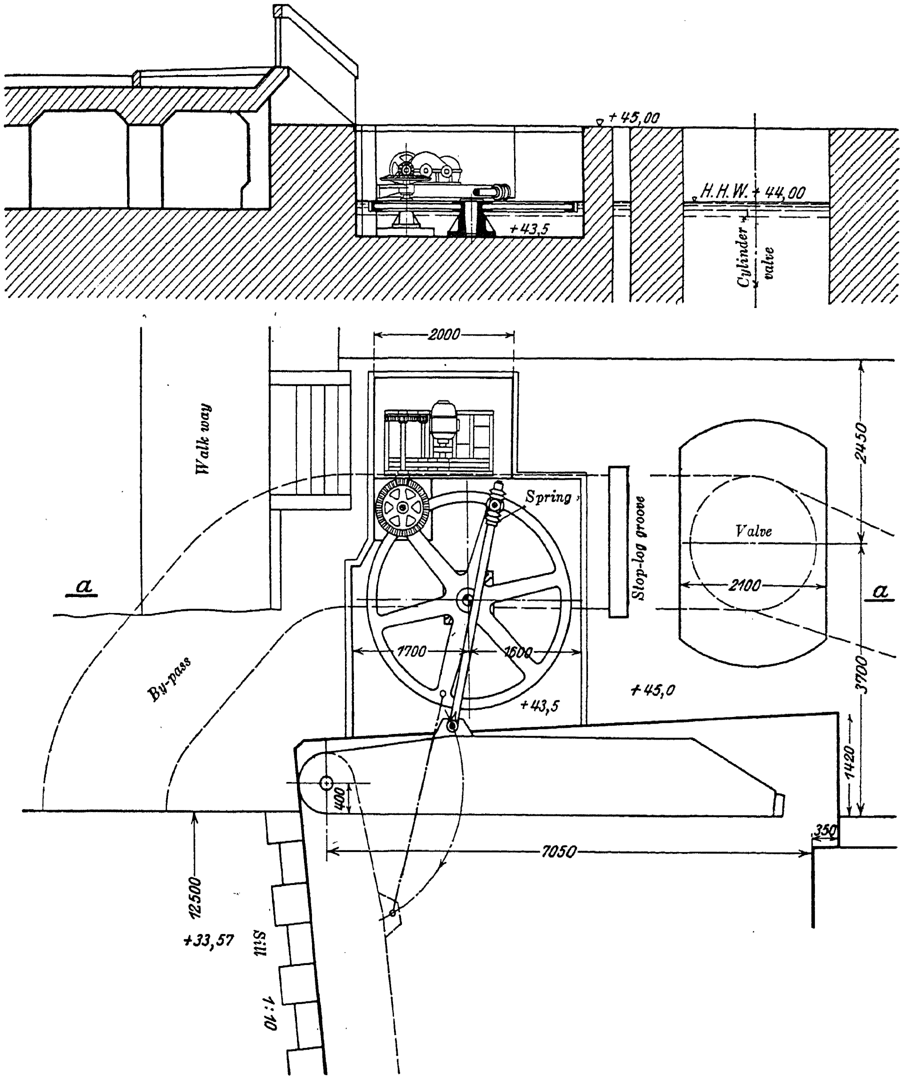


Fig. 407. Mitering-gate drive with crank wheel. Designed by *Freund-Starkehoffmann-Maschinen A.-G.* for the Weser descending lock at Minden

ment was patented and installed in the locks of the Elbe-Trave Canal, along the Oder, and in the old harbor lock on the right bank of the Weser

at Minden. It is not probable that this type of device will continue to be employed where electric current is available.

Nyholm's method employs so-called plunger plates (Fig. 410). Two

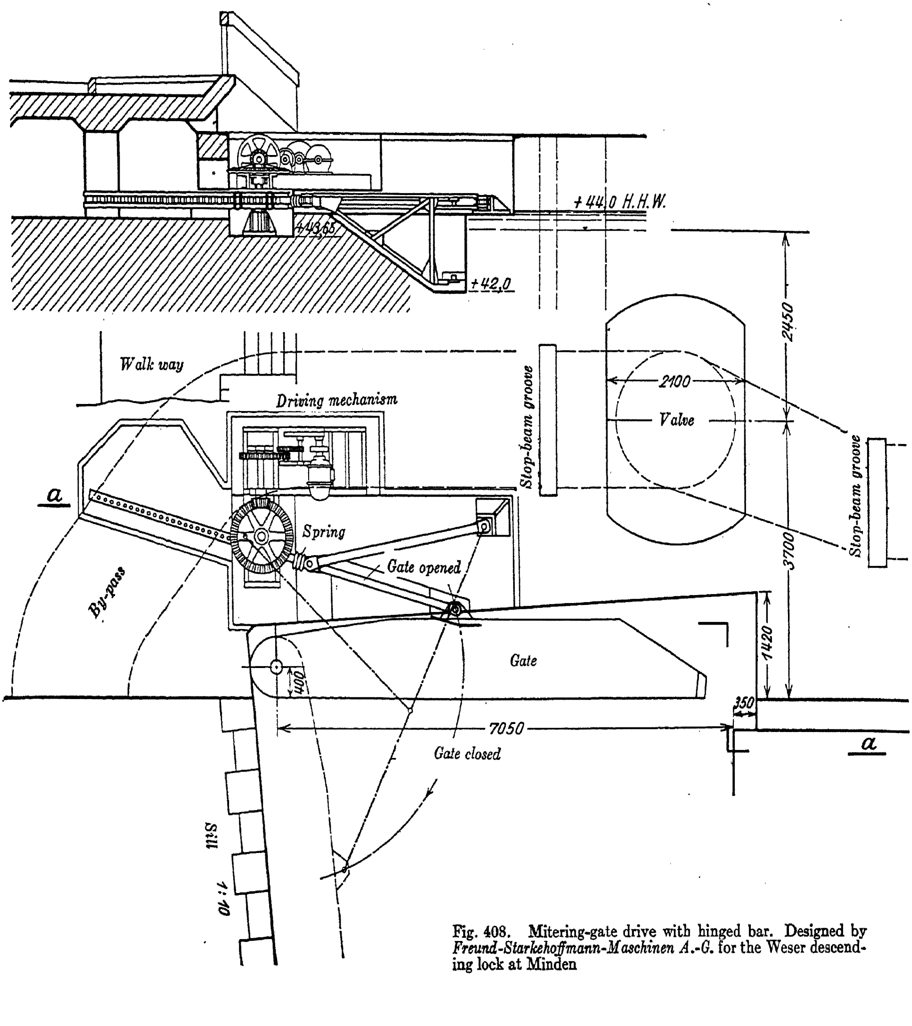


Fig. 408. Mitering-gate drive with hinged bar. Designed by *Freund-Starkehoffmann-Maschinen A.-G.* for the Weser descending lock at Minden

shafts are arranged so that they are subject to the headwater pressure above. By means of a four-way stop cock, each shaft can be connected either with the headwater or the tail-water. If the bottom of one

shaft is connected with the tail-water, the other is joined to the headwater. Thus the first plate receives the entire pressure of the headwater from above, and the plate is forced to move downward. The other plate is under headwater pressure simultaneously from above and below; hence, since it provides no resistance to motion, it is drawn upward by the first plate. By changing the position of the four-way stop cock, the movement is reversed. The movement of the plates actuates a cogwheel which propels the connecting rod. The same type of drive can be employed to raise sluice gates. The Nyholm arrangement has proven very satisfactory in the locks at Hemelingen. When using this layout for operating the gates of the tail-bay, the headwater must be conducted to the tail-bay through a pipe, and the head-bay must be connected with the tail-water by a pipe. These pipe lines must be very carefully constructed and require a greater investment than would be required with electrical equipment serving the same purpose. The invention, interesting in itself, is of importance where electricity is not available. If electric power is at disposal, it is inferior to electrical equipment.

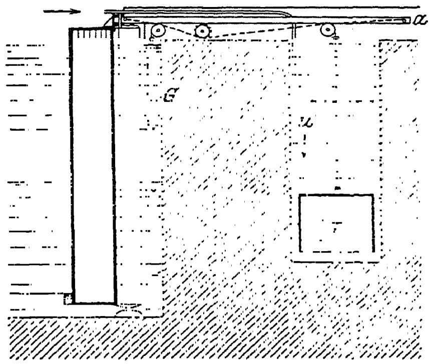


Fig. 409. Hotopp drive arrangement

Franke's method employs two floats which are connected by a rope over pulleys. The method was applied to the lock at Meppen to operate a tumble gate. However, it can be used just as readily for mitering gates. The shafts may be connected with the headwater or tail-water by means of gate valves. The arrangement is similar to that at Hemelingen, except that the plates are replaced by floats. The advantages and shortcomings are the same for both. Franke's method may also be used with only one shaft. In such an arrangement, the counterweight dips into the headwater or tail-water (Fig. 411).

It has been suggested to undertake actuating gates with ship propellers driven by electric motors. The units would be built into the gate. This procedure is not to be recommended because it would not be possible to take up the wave thrusts with enough certainty. A wave, striking the closing gate, might lead to damage which would be entirely avoided in case elastic connecting rod drive were used. Furthermore, the propeller is not capable of pressing the gates tightly together.

In conclusion, attention is called to the interesting comparison of the designs for the gates of the new locks on the Weser at Minden. The

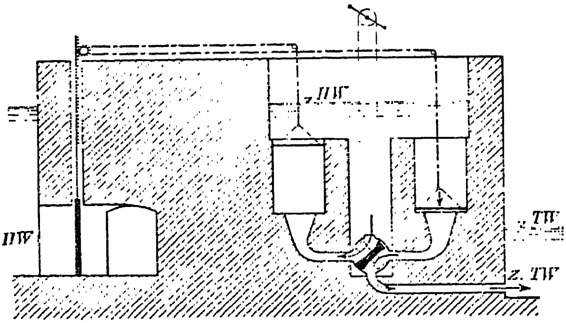


Fig. 410. Method of operation proposed by Nyholm

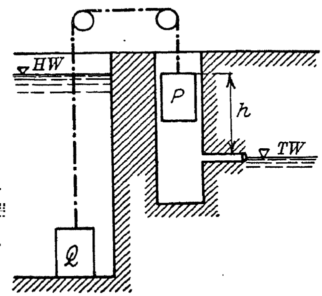


Fig. 411. Method of operation proposed by Franke

three designs of the Freund-Starkehoffmann-Maschinen A.-G. (Figs. 405, 407, and 408) show the differences that may result with different methods of actuating the same gate.

c. Tumble Gates

1. Tumble Gates with Horizontal Axes

The tumble gate is used a great deal in the head-bays of inland locks but is not used along the sea. The gate consists of a shield as wide as the lock entrance and extending vertically from the port sill to above the headwater level. Since the more recent canal locks are about 12 to 13 m. (about 39 to 43 ft.) wide and have a sill depth of 3 to 4 m. (10 to 13 ft.) below the headwater, gates of about 3.3×12.3 to 4.3×13.3 sq. m. surface are required, which are relatively lightly stressed in the vertical. Tumble gates are now generally inclined toward the water side, so that their weight acts favorably for lowering them when the water pressure is of equal magnitude on both sides. In order to avoid damage by ships crossing a tumble gate, a recess should be provided for the gate in the floor. The recess must be of such depth that no part of the gate protrudes above the floor level. The older gates of this type, for example those built by Mohr for the locks at Fuerstenberg on the Oder-Spree Canal, were made of wood. They were raised and lowered by a chain. Mohr used a stilling basin at the head-bay of such installations. More recent and better plans of this nature were made for the locks of the Hohenzollern Canal at Niederfinow, for the shaft lock at Minden, and others.

Probably the best design for tumble gates consists of two strong

horizontal girders joined by uprights. Sheathing is usually put on both sides; diagonal bracing is then dispensable because the metal cover furnishes sufficient stiffness. Using metal sheathing on both sides simplifies providing an air chamber. The air chamber must be accurately designed in order to determine the desired state of equilibrium correctly. It is usually advisable not to continue the air chamber throughout the entire width of the gate, but to distribute chambers between certain uprights. The arrangement should be made entirely dependent upon the design of the gate.

In Fig. 412, if A is the buoyancy, D the pressure on the gudgeon, and G the weight of the immersed gate without buoyancy, then the gudgeon pressure is $D=G-A$. If the distance from the pivot of the weight and buoyancy, respectively, is g and a, and if the friction moment of the bearing is $M_D=\mu \cdot D \cdot r$, in which r is the radius of the journal, then

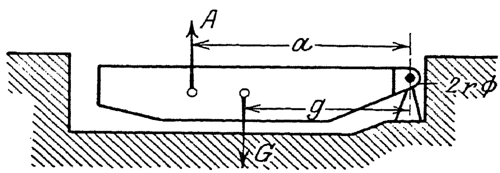


Fig. 412. Force system obtaining when raising a trap gate

$$A \cdot a=G \cdot g+\mu \cdot D \cdot r=G \cdot g+\mu Gr-\mu Ar.$$

Then

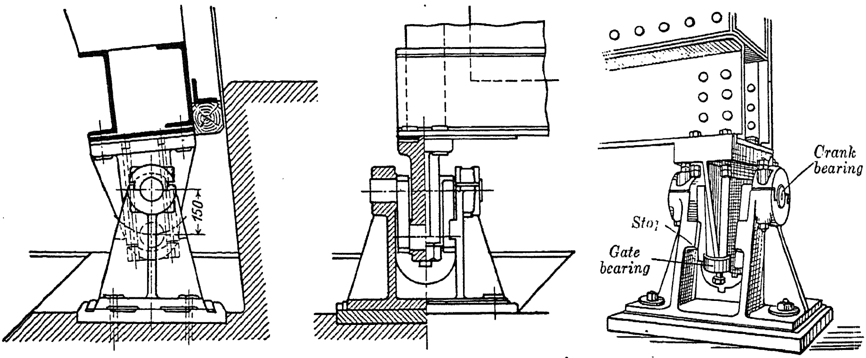
$$A=\frac{G(g+\mu r)}{a+\mu r}.$$

From this it follows that A is the smallest when a is greatest. Accordingly, the air chamber should be placed as high as possible, but low enough so that it remains under the water surface when the headwater is lowest. With this arrangement, the air chamber also receives the least water pressure as long as the gate is raised. The use of the air chamber is to be recommended for tumble gates, notwithstanding the contrary for mitering gates, because during every complete cycle of operation the chamber emerges (once) entirely from the water, and therefore can be readily kept free of water at all times. Merely a simple double stopcock arrangement is necessary; it can be served by a small float along the gate. The float opens the lower discharge valve and the upper vent cock whenever the water in the chamber has fallen sufficiently and closes the stopcocks during the filling of the chamber. If automatons of this kind are not considered dependable, the stopcocks may be hand operated by a lock attendant from time to time.

Formerly the lower bearing was often of very primitive design. Buchholz introduced the best arrangement.[1] He employed a double

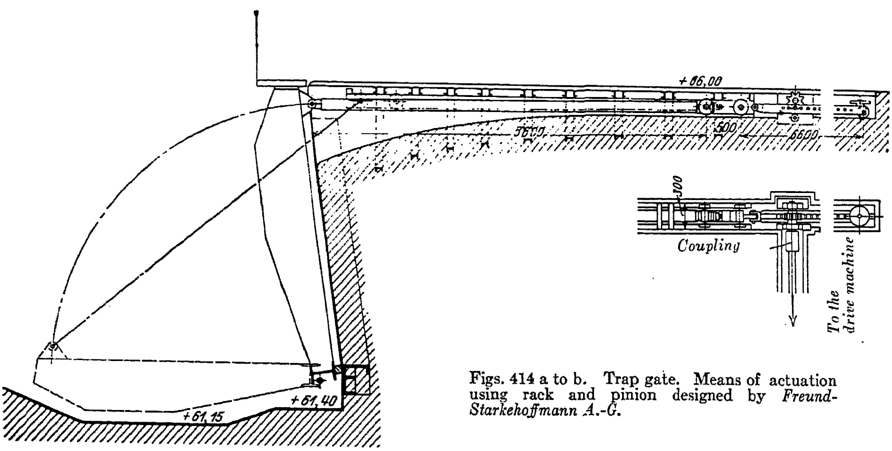
[1] *Z. V. d. I.*, 1913, p. 1809.

bearing in which the actual gate bearing rests on a U-bent shaft which is rigidly fastened at both ends (Figs. 413 a to c). This arrangement allowed the possibility of displacing the gate bearing as in the case of the swinging



Figs. 413 a to c. Swinging bearing for the gate proposed by Buchholz
a and b. Cross-sections c Elevation

supports for mitering gates. When the gate is closed, the water pressure forces the gate against the stop; but if a piece of wood is wedged between the sill and the gate, the U-shaft revolves backward, thus avoiding destruction of the bearing. At Minden, such an arrangement proved itself somewhat weak. It is advisable to construct the bearings heavy

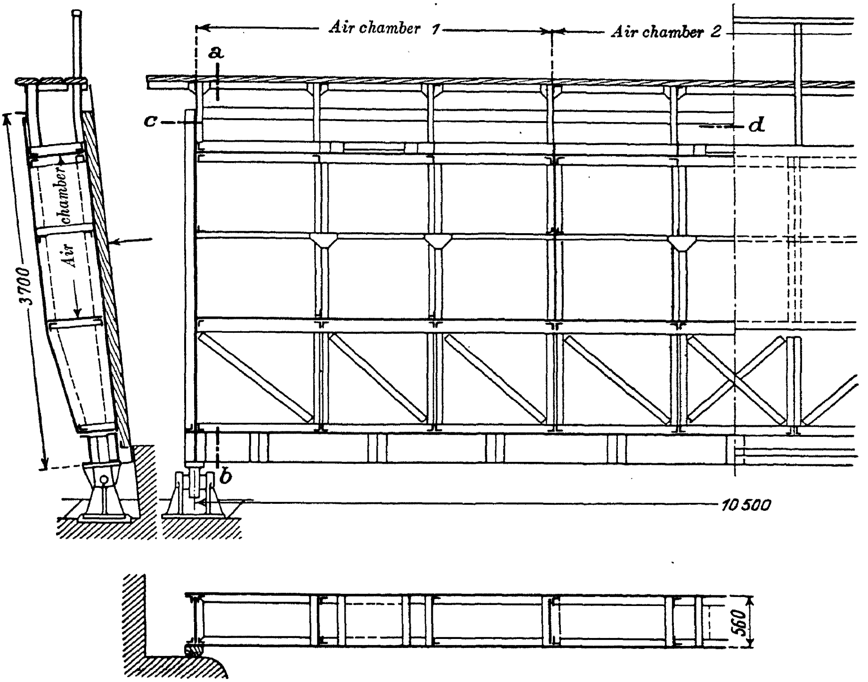


Figs. 414 a to b. Trap gate. Means of actuation using rack and pinion designed by *Freund-Starkehoffmann A.-G.*

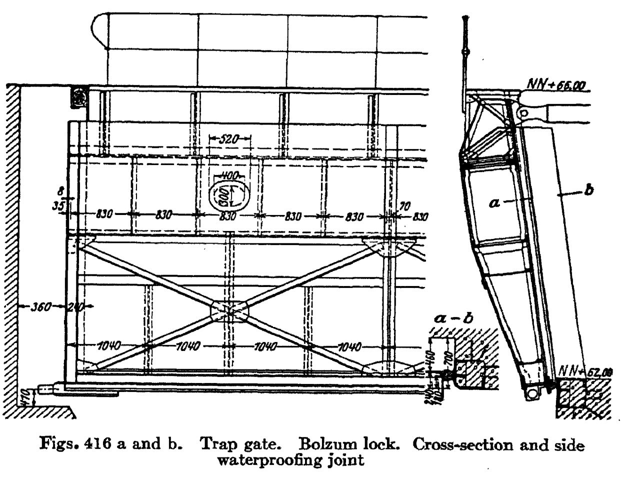
enough to absorb the stresses which can not be accurately predetermined. Movable bearings should be used exclusively.

Buchholz also introduced an innovation in gate drive by using a rigid

connecting rod (Fig. 414). Chain drive had made good but was cumbersome. Buchholz used a connecting rod which is moved by a crosshead operating close to the upper edge of the wall. The connecting rod is



Figs. 415 a to c. Trap gate of the lock at Schandau
a Section a–b b Elevation c Section c–d

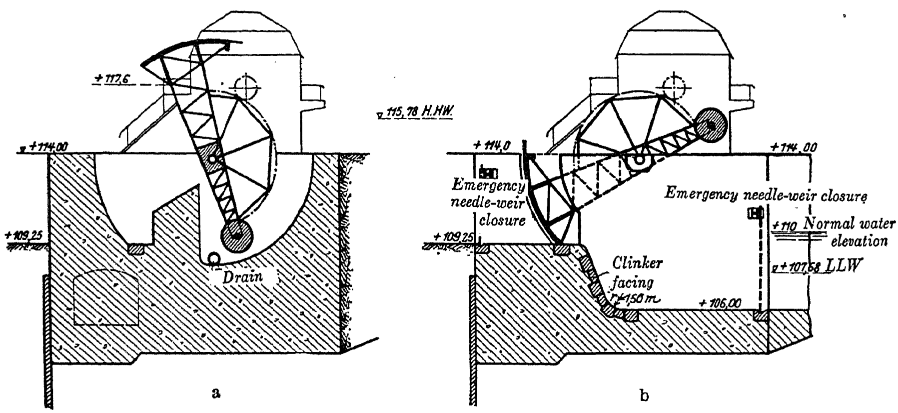


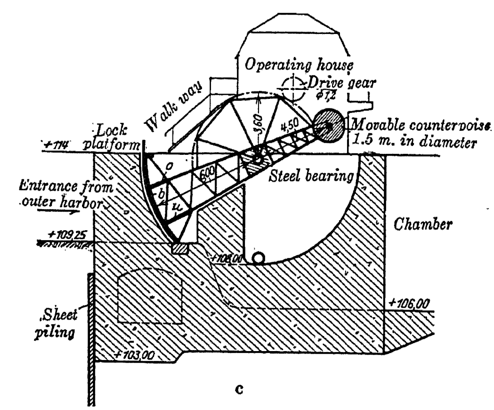
Figs. 416 a and b. Trap gate. Bolzum lock. Cross-section and side waterproofing joint

hinge-connected at both ends and made elastic by the insertion of a spring. The crosshead is moved back and forth by a second connecting rod which moves horizontally in a slot in the wall back of the gate stop. The slot must be curved downward so that the rod does not come in contact with the wall. Designs which have been put into effect are shown in Figs. 415 and 416.

2. Tumble Gates with Vertical Axes

These gates are used but seldom. In the form of single-leaf gates, they close off the lock chamber in a very simple manner, but require considerable lengthening of the bay and, therefore, are usually uneconomical. This led to the invention of mitering gates. The two-leaf tumble gates were then developed. Since a middle column in the center is impossible,





Figs. 417 a to c. Mitering gate to the Gröschel lock at Breslau
a Gate in open position
b Section through lock chamber. Gate closed
c Section through gate chamber. Gate closed

the gates must close on each other in a groove. Such gates enter into consideration only for shaft locks where both an upper and a lower sill can be provided. It is advisable to give the gate stops a slight slant for these gates so that the middle joint is made waterproof by a slight amount of mitering pressure.

d. Segment Gates, Lift Gates, and Cylinder Gates

1. General

These three kinds of gates are distinguished by the fact that the gate is raised entirely out of the water when being opened. They have the disadvantage of having to be raised above the lower limit of the overhead clearance required. Hence they are not applicable to sea locks. Their use may cause difficulty in river locks if great differences exist between the normal headwater level and the highest navigable water level. These gates are applicable primarily to inland canal locks. Moreover, lift gates are suitable for use in the lower end of shaft locks. The discussion of designs in the chapter on "Weirs" concerning segment

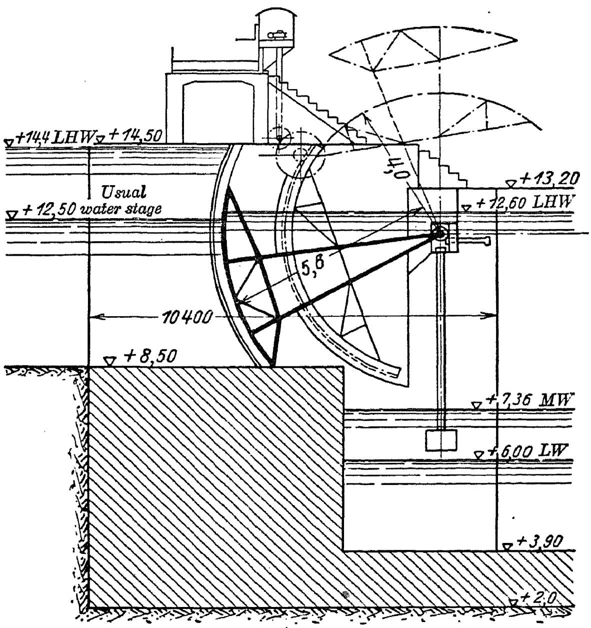


Fig. 418. Grosswohnsdorf lock in East Prussia on the Alle River
Segment gate in the head-bay. Breadth 7.5 m

weirs, sluice weirs, and cylinder weirs, is also applicable here. To avoid repetition the reader is referred to the detailed treatment of these structures under "Weirs."

2. Segment Gates

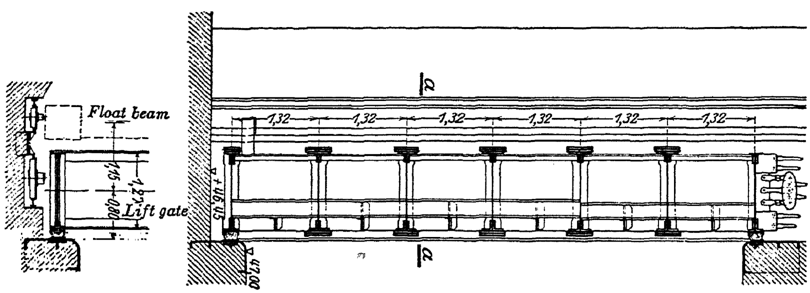
The style of structure is determined by the overhead clearance required above the highest water level. The revolving bearing is placed as high as possible so as to require a minimum length of the segment

arms. The arms must recede completely into recesses, so that after the gate is raised no part extends beyond the lock wall. A special sill is not necessary; an iron strip embedded in concrete, on which the lower water-seal rests bluntly, suffices. The construction is similar to that of weirs. The counterweights may also be arranged in the same manner. Since the arms lie in lateral recesses, there is also sufficient space to permit the installation of balance beams with counterweights. The damming shield of the gate may be equipped in such a manner that when the gate is raised, it forms a passage across the bay. Figs. 417 to 418 show examples of segment gates.

3. Lift Gates

As long as lift gates are to be opened only when the water level on the two sides is equalized, they may be constructed as simple sliding gates. But if they are to be used also for filling and emptying the chamber, they must be built as roller gates. Two types of lift gates are to be distinguished: first, those at the ends of usual locks; secondly, those serving as closures of the lower ends of shaft locks.

The lift gates of ordinary locks resemble large weir sluice gates. They require superstructures above the lock bays, into which they can be raised. Although lifting is to take place only after the equalization



Figs. 419 a and b. Lower gate of the Anderten lock (Hanover, Germany) in section

of the water level, guide rollers must be installed, which can be readily constructed since they have almost no stress to sustain. As to the steel structure, lift gates do not differ from tumble gates; in both types, large damming shields are arranged to close the opening and are provided only with a sill and side bearing surfaces. In this case the best design is also obtained by the use of two heavy horizontal girders with uprights between them. The gates of shaft locks are materially heavier

than those of ordinary locks but have the advantage that all four sides are supported. Lift gates are provided with counterpoises which move

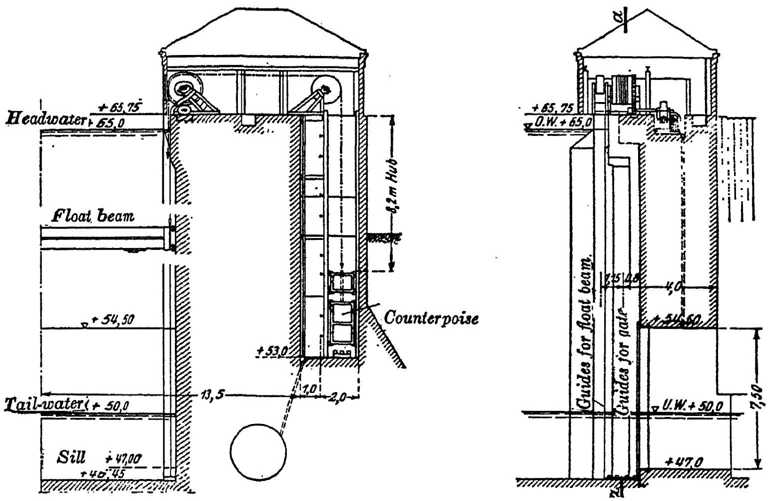


Fig. 420. Gate and float-beam installation of the lock at Anderten

in recesses at the side of the gate. The gates are readily protected by wooden fenders which float at the water surface, and they are raised together with the gate (Figs. 419 and 420).

4. Cylinder Gates

Although the cylinder gate has the same qualifications as segment gates and lift gates, it has seldom been used in locks. If correctly designed, the gate should not become any more expensive than the lift gate, although probably more expensive than the segment gate. Inasmuch as it must be raised to above the clearance required for ships, a special superstructure is required over the head-bay to support the roller tracks. No superstructure is required at the tail-bay, provided the lock lift is greater than the clearance required above ships. The cylinder gate and the segment gate may both be actuated from one side. This possibility does not provide any particular advantage over the lift gate, as the latter requires a superstructure regardless of the manner of drive, and the superstructure can readily be arranged to support operating equipment in the center of the span.

e. Pontoons

1. Floating Pontoons

The original type of floating pontoons was of a shape similar to that of ships, the design being based entirely upon the rules of shipbuilding. They are floated into position and then sunk by allowing water to enter. In order to provide a satisfactory water-seal, the pontoon is supplied with a bearing strip around its circumference. The old type of pontoon is not suitable for locks because of the excessive amount of time required for operation. However, floating pontoons have proven very satisfactory for dry docks. They are also used for locks, in which case one end is held in position by a hinge so that the structure operates as a single-leaf gate.

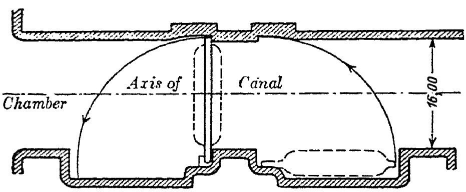
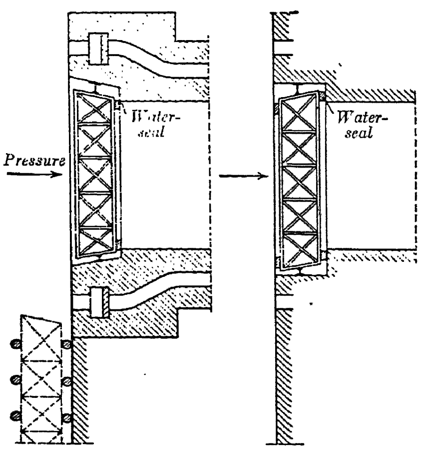


Fig. 421. Swing gate of the east lock at Tancarville Plan view

A vertical axis is located in a groove in the wall just as for vertical tumble gates. The recess must be very deep because of the thickness of the pontoon. Furthermore, the lock bay must be made very large because the pontoon is longer than the clear breadth of the chamber (Fig. 421). Newer floating pontoons differ greatly from the ship form originally used; they are of box shape and have the water-seal and bearing on the front plane of the caisson (Figs. 422 and 425). The floating capacity of these pontoons is greater than the older type, since the caissons displace a much greater quantity of water near the bottom. For dry docks, such floating pontoons are to be recommended in preference to sliding pontoons; particularly if the dock lies in a harbor where sea motion is not serious. In the latter case, the opening and closing can take place relatively rapidly. Sliding pontoons are not necessary in docks because the gate is moved very infrequently, the ship generally being in dry dock days or weeks at a time.



Figs. 422 a and b. One-way and two-way movable pontoon gate

The height of the metacenter is of particular significance for these pontoons. Figs. 423 and 424 indicate the method of finding the metacenter. The height of the metacenter is the distance between it and the center of gravity of the pontoon. This height should be at least .4 m. (1.3 ft.), preferably greater if possible. The method of increasing this distance consists in designing a light structure, inasmuch as a heavy floor ballast will then be required to sink the pontoon sufficiently. The floor ballast causes the center of gravity to be moved downward and thereby increases the metacentric height. Fig. 423 shows an unstable pontoon; Fig. 424 a stable one.

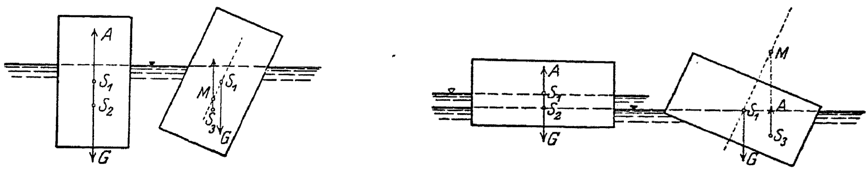


Fig. 423 Figs. 423 and 424. Metacenter Fig. 424

2. Sliding and Rolling Pontoons

A rectangular shaped pontoon naturally lends itself to being moved in a longitudinal direction on a sliding track or rollers. The time required for opening and closing a gate can be greatly diminished thereby, the amount of time required being reduced to approximately that necessary for operating large mitering gates. Sliding gates necessitate the construction of a special gate chamber protruding from the side of the lock (Figs. 425 and 429). Such pontoon chambers are especially endangered points of locks. Sliding pontoons at present are principally of historical interest; they have been used frequently but are no longer to be recommended. Certain disadvantages which occur with this system have largely been avoided by the use of rollers. In experiments made at Emden, reported by Zander, it was found that, under normal operation of sliding pontoons, all greenheart strips were worn down 1 cm. (.394 in.) within one month. This was due to the sandy silt which deposits in this vicinity. Thus sliding pontoons should be used only where the water is free from gritty material.

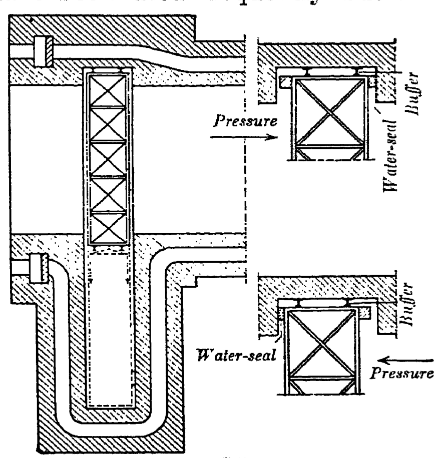


Fig. 425. Two-way sliding gate (pontoon)

In addition to the danger of abrasion, the sliding pontoon has the further disadvantage that it operates jerkily. This condition presented itself at Kiel and was found to be the result of too light a track loading. Movement of the gate was facilitated by forcing the water from the air chambers so that a pressure of only about twenty tons was exerted against the sliding track. It was found that pressure waves created by ships as they passed the gate rhythmically varied the buoyancy; the pontoon thereby underwent a horizontal rocking motion similar to the pitching of a ship. Consequently, very irregular stresses were incurred in the operating chains. Some of the equipment had to be rebuilt, and the pressure upon the slides had to be increased to fifty tons. All sliding pontoons are now constructed in such a manner that they may also be used as floating pontoons.

3. Types of Pontoon Structures

a. Steel Pontoons

The pontoon should be built of horizontal girders with uprights spanned between them. Since a pontoon must be quite thick [4 to 8 m. (13 to 26 ft.)] in order to provide the necessary ability to float, the girders possess great flexural strength. It is advantageous to construct the entire floating pontoon as a box girder. Because of the great height, such a girder must have diagonal stiffening. The latter is analyzed as a truss. The verticals serve not only to resist the water pressures against the sheathing, but also as transverse stiffeners; they are therefore designed as members of a vertical truss system. In the longitudinal direction the necessary stiffening is obtained by means of the outer sheathing, which is fastened to both sides of the gate. This sheathing, in conjunction with the horizontal girders and vertical posts, provides enough stiffening action to make longitudinal diagonals unnecessary.

In the proximity of the sill where low air chambers are used, a bottom girder is usually dispensable; the uprights then bear against the box girder (air chamber) at the top and against the sill at the bottom. In any case, a study should be made to determine whether the pressures exerted against the sill are too large, and whether a better solution is obtainable by providing a bottom girder. If the latter procedure is adopted, it is advisable to extend the lower ends of the uprights downward and provide them with enough flexibility so that sufficient pressure is exerted against the sill by the bottom of the pontoon.

Pontoons are frequently constructed trapezoidal in elevation so that they may be more readily lifted from the recesses by being buoyed upward (Figs. 426 to 438). The same facility of operation is offered if

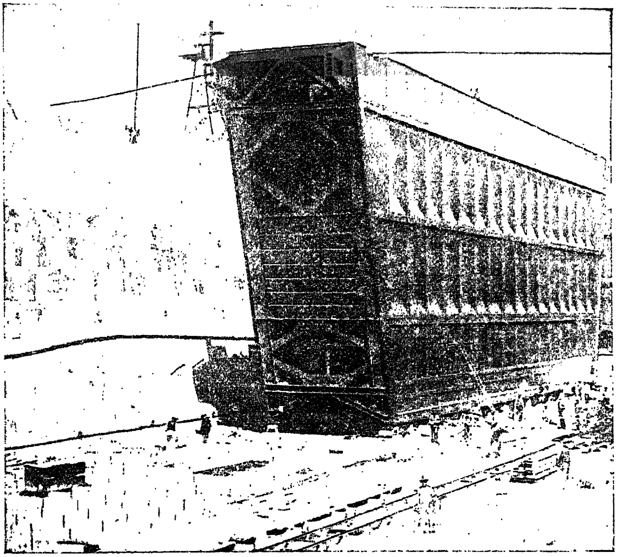


Fig. 426. Dock gate. Puerto-Militar

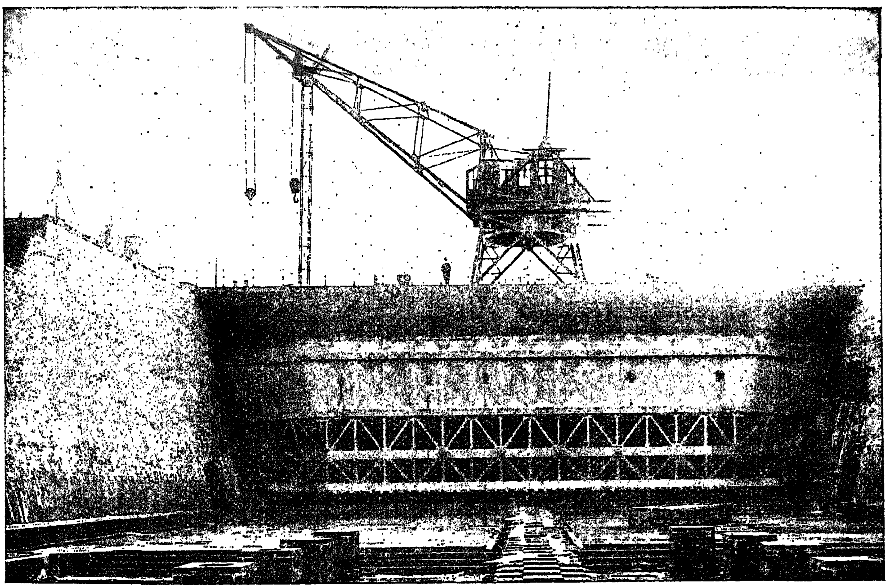
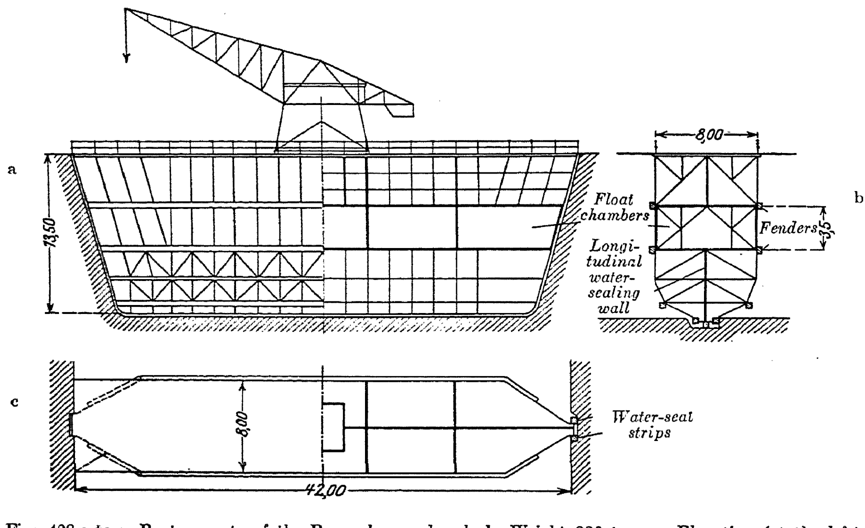


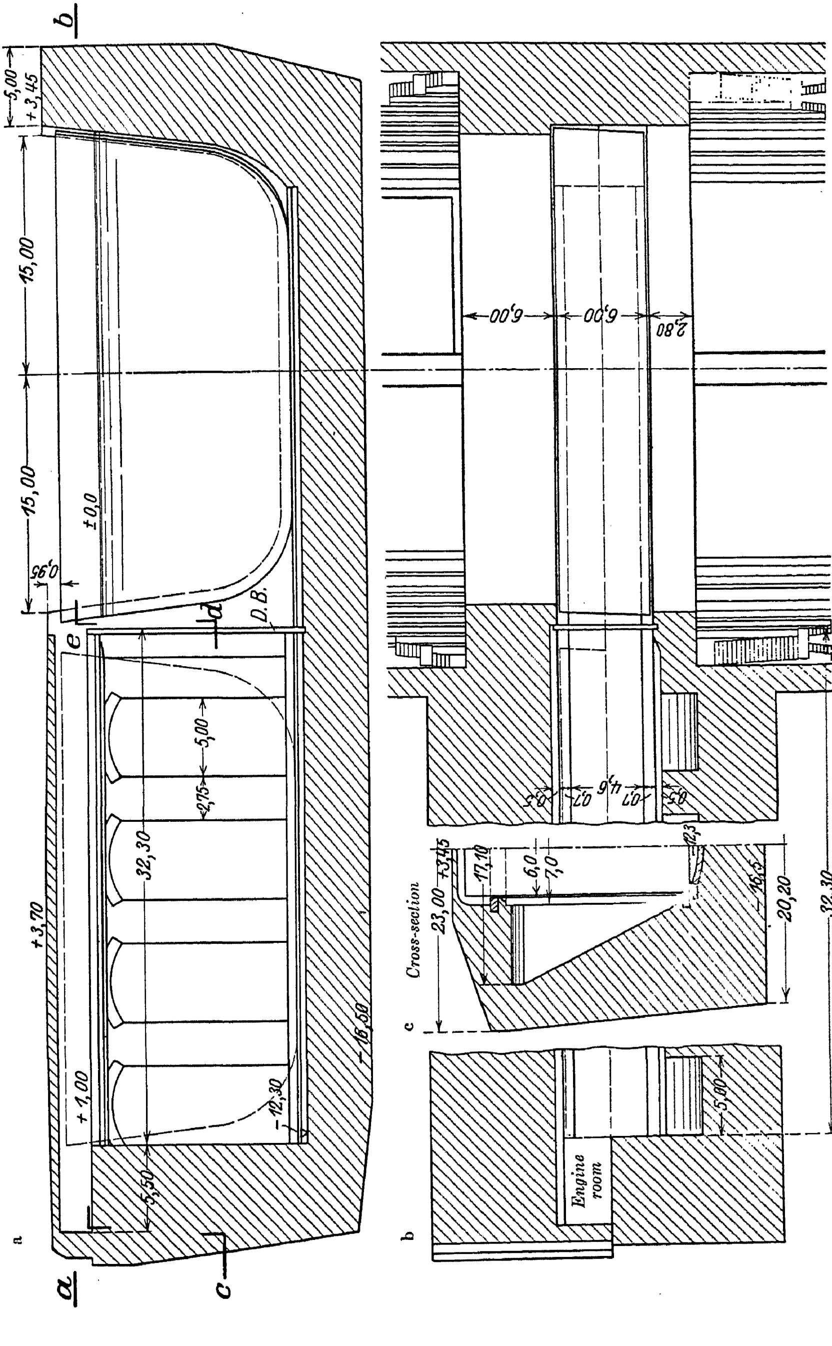
Fig. 427. Pontoon gate at Bremerhaven
Clear breadth 40 m. at the top and 30 m. at the bottom, height 13.60 m

the pontoon is rectangular in elevation, provided the groove (not the gate stop) is of trapezoidal shape. The rectangular form of gate is advantageous, especially for locks, so that this type usually receives preference. To facilitate moving the gate out of the recess, the pontoon is made trapezoidal in plan (Fig. 422 a) as first proposed by the English engineer, Kinipple; or still better, in the form of a rhombus. The latter was first used for gates of dry docks 5 and 6 of the German Marine at Kiel (Fig. 422 b). The form proposed by Kinipple has the disadvantage of not being symmetrical, so that the gate can be used only in one direction, while the form used at Kiel is applicable to damming water on either side. The pontoon is moved from its closed position by turning it about a vertical axis and simultaneously displacing it longitudinally. Waterproof transverse partitions in the pontoon should be avoided, so that the water, which must enter the chamber when the gate travels outward and must leave the chamber when the gate travels



Figs. 428 a to c. Pontoon gate of the Bremerhaven dry dock. Weight 880 tons. a Elevation (at the left), longitudinal section (at the right) b Cross-section c Plan view (at the left) and horizontal section through pontoon (at the right)

inward, can find its way through the pontoon. Kinipple avoided this difficulty by placing the rear end of the chamber in communication with the harbor by means of a culvert, which allows movement of water to and from the chamber. In the case of one arrangement in which the gate is moved directly by water pressure (Wilhelmshaven, suction and pressure movement), impervious transverse walls are required. In this



Figs. 420 a to c. Kiel dry dock. Pontoon chamber of Dock 5
a Longitudinal section through pontoon chamber b Section a–b above and section c, d, e, b, below c Cross-section of wall

case the water is forced in and out of the pontoon chamber by means of large pumps (M.A.N. propeller pumps[1]).

Attention is also called to the experiences at Kiel concerning the installation of a subfloor. The latter, which lies near the bottom of the pontoon, became filled with silt, mussels, and the like, to the extent that it became difficult to move the pontoon. The floor metal was discarded and replaced by a lattice work girder. Figs. 428 a to c presents an example of a well-designed pontoon (inside gate of the Kaiser lock at Bremerhaven). The design in many respects serves as a pattern to the present day. The longitudinal diagonals are dispensable. Further figures are shown of the pontoons at Emden and Puerto Militar which were constructed by M.A.N.

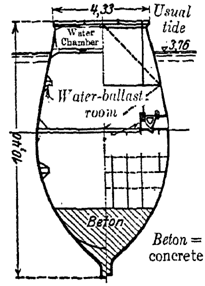
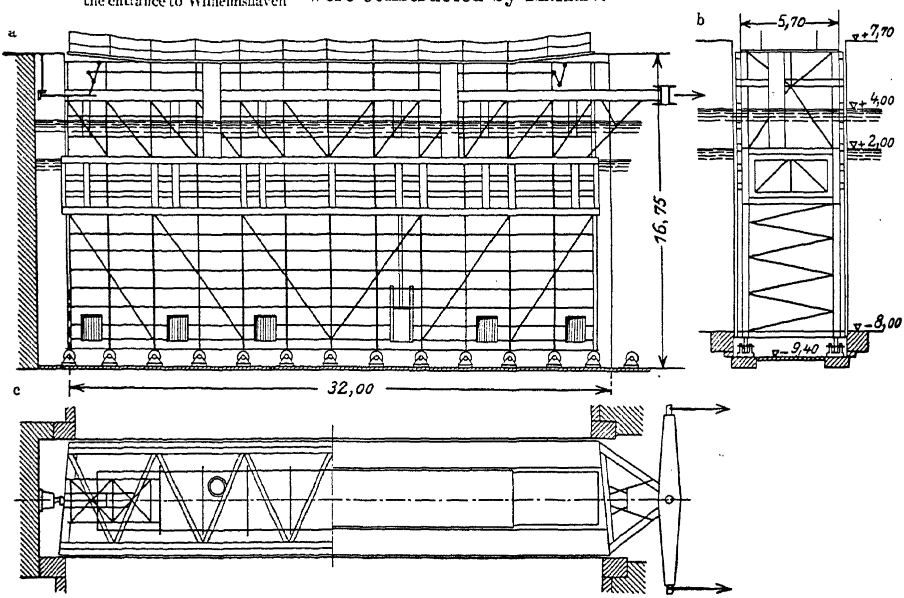


Fig. 430. Pontoon gate of an antiquated form. Inner bay of the entrance to Wilhelmshaven



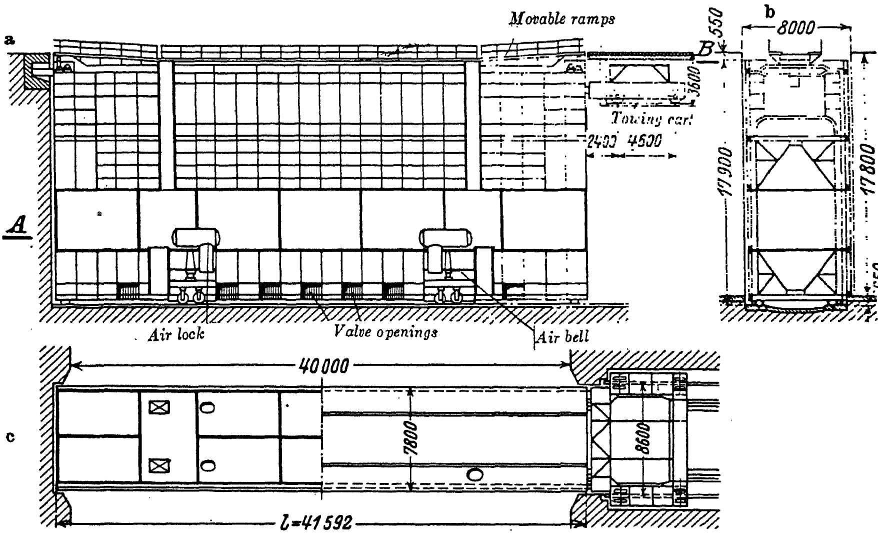
Figs. 431 a-c. Sliding gate on rollers. Kaiser lock at Bremen
a Front elevation b Cross-section c Plan view

Inasmuch as dry docks are constructed with the walls stair stepped on the inner side, the trapezoidal form of pontoon (Figs. 426 to 428) is more advantageous than the rectangular form, particularly if only floating pontoons are considered. Floating the pontoon in and out of position is very much facilitated by using the trapezoidal form.

[1] The M.A.N. propeller pumps are used for large quantities pumped under low heads.

β. Roller Installations

One of the oldest roller installations is that for the Kaiser lock at Bremerhaven. Here the rollers [80 cm. (32 in.) in diameter] were fastened to a slide so that the pontoon was entirely smooth along the bottom. The arrangement is antiquated because, though it proved entirely satisfactory, it is too expensive. Obviously if the rollers are fastened to the pontoon, comparatively few of them are necessary. Similar considerations at Bremerhaven led to placing the disks of the gudgeon bearings of mitering gates in the gate. It was feared that



Figs. 432 a to c. Sliding gate of the new sea lock at Emden. Total weight 1,200 tons

a Longitudinal section b Cross-section c Section A (at the left) and plan view (at the right)

wheels rolling over fixed rails would meet obstructions which would hinder their motion, while a cable or piece of wood might lie between wheels fixed in position without causing damage. This disadvantage may be overcome by arranging a well-designed clearing apparatus for the path of the roller.

At Davenport, an intermediate between the sliding and rolling type of pontoon was constructed. Here vats are provided on both sides, and between them a wheel which rolls on a center rail. When in motion, the front wheel is forced down by means of hydraulic pistons and the rear wheel is raised so that the pontoon runs on the wheel in the front

and on the vats in the rear. The characteristic jerky motion is eliminated thereby. This arrangement, however, is not of consequence because if one wheel is admissible, several might just as well be used.

More recent installations in other harbors have been provided with four roller sets of two or more each. But it was always feared that these mechanical parts which are continually under water might get out of order. Consequently, Behrend in Wilhelmshaven and Zander in Emden surrounded the wheels by metal housings in such a manner that the wheels might be made accessible by forcing air into the housing. This is an ingenious arrangement but somewhat intricate (Figs. 432 a to c). New designs, therefore, such as on the Kaiser-Wilhelm Canal, have been more advantageous. The best solution may possibly be found by a scheme in which the rear roller nest is mounted above the water surface so that the pontoon rests upon a car which rolls over the top of the pontoon chamber, while the front end of the pontoon rests upon a roller car which is replaceable. Such a pontoon might be arranged so that it can be tilted enough for the front car to be lifted out of water. The car may then be removed by means of a floating crane and replaced by another car. The gate at Brunsbüttel is of this type, whereas the gate at Holtenau possesses two underwater cars which can be replaced by others with the aid of cranes. One factor which must be kept in mind in connection with all roller pontoons concerns the lateral displacement of the pontoons upon the roller axes. The pontoon must have plenty of playroom when in motion to prevent dragging against the stop wall and sill; however, when once in place, the excess water pressure from one side must cause it to be moved snugly against the stop bearings and sill (Fig. 427). Consequently, lateral displacement of the gate upon the axes of the wheels and lateral displacement of the wheels upon the rails are indispensable requirements. If no provision is made for this lateral displacement, rupture is unavoidable.

γ. Operating Appliances

The movement of a sliding pontoon is effected by means of a balance beam (Fig. 431) which is attached to the center of the top of the pontoon in such a manner that it may be uncoupled. The arrangement allows for pulling the gate sidewise through the pontoon chamber either by means of endless chains or racks. The arrangement with cables (dry docks at Palmers Wharf in Newcastle on the Tyne with open chamber) or chains (Bremerhaven, Kiel) has proven satisfactory, but various elementary shortcomings must still be eliminated. In order to avoid the unequal pull resulting from different degrees of distortion of the

cable, Kinipple arranged the layout as indicated in Fig. 433. It possesses four fixed rollers on the walls of the pontoon chamber and two on the balance beam over which a single cable operates. The cable over the pontoon rollers incurs practically no movement, and can be held taut by a special arrangement.

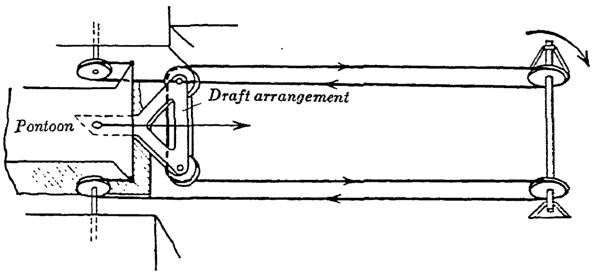


Fig. 433. Kinipple's mode of operation for a sliding gate

Nevertheless, a rigid drive consisting of racks or the like is preferable to other devices. The racks may either be movable and propelled by means of a pinion at the rear end of the pontoon chamber (Kaiser-Wilhelm Canal), or they may be fixed to corbels in the pontoon chamber and the operating mechanism placed on a special car to which the pontoon is fixed (Emden, Figs. 432 a to c). By using a movable rack, which possesses advantages in that the entire mechanical equipment can be located in a closed room, rack tunnels are required, the racks being run into these on rollers when the gate opens. These tunnels lie directly below the ground surface and should be accessible from above.

δ. Comparison of Mitering Gates and Sliding Gates[1]

Extensive investigations have shown that in the case of gates loaded only from one side, mitering gates are cheaper than sliding gates. On the other hand, if subsequent laying dry of the bays is indispensable, sliding gates are likely to be cheaper in large layouts. If the operation must be in both directions, the sliding gate is the cheaper, the amount of saving increasing directly with the lock breadth. The cost of a sliding gate is usually not much greater than that of a pair of mitering gates, but is invariably less than that of two pairs of mitering gates.

Opening and closing sliding gates against a head of water or against a current is not dangerous, and can be accomplished more safely than with mitering gates. The sliding gate is a much more rigid body in every respect than the mitering gate and therefore is less influenced by flexural stresses. There is also a higher degree of certainty in the operation of sliding gates.

The floating gate (pontoon) is usually the best type to use for dry

[1] Regierungsbaurat W. Groth, *Doctorate Dissertation*, Vor- und Nachteile des Schiebetores für Schiffsschleusen usw., Technical University of Hanover.

docks, being superior to the sliding gate and usually also to the mitering gate. The construction of sliding gates for dry docks in the German Marine may be considered too expensive an arrangement.

f. Special Construction Types

1. Hotopp Tumble Gate

Hotopp locks are operated by means of compressed air. Hotopp used this source of power very ingeniously to move his tumble gates. The gate is provided with an air chamber just below the water surface; the chamber receives compressed air which is allowed to enter the gate freely when the latter is in a lowered position (Fig. 434). The air dis-

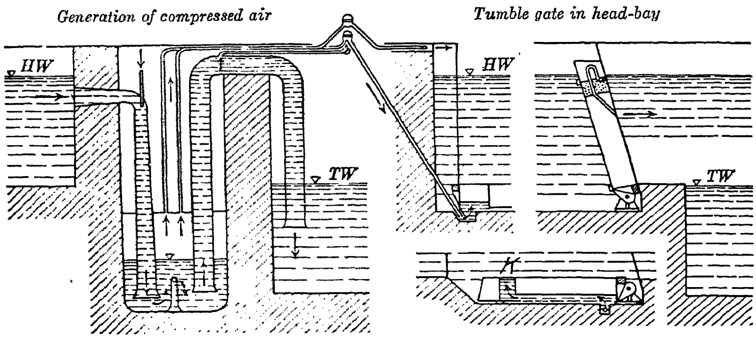


Fig. 434. Hotopp trap gate

places the water, thus causing the gate to rise automatically. By means of a siphon the air chamber may be allowed to become filled with water again so that the gate can be lowered by virtue of its own weight. The arrangement is very well devised, operates well, and is to be recommended in localities where electric current is not available.

2. Fan Gates

Figs. 435 and 436 indicate the design of a fan gate. The concept involved is the same as that of types proposed by Nyholm, Franke, Hotopp, and others; namely, to operate the gate by making use of the difference in water stage at the two ends of the lock. A culvert must reach from the head-bay to the tail-bay so that the tail-bay is continually supplied with the static head available between the pools. The inner leaf, which is rigidly connected to the mitering gate, can be subjected to either headwater pressure or tail-water pressure by correspond-

ing settings of the valves connecting the by-passes. Brennecke recommends the use of these gates. The structures erected within the last generation, however, indicate that this scheme is not very popular. If mitering gates are to be used, operation by some sort of modern appliance such as a connecting rod is simpler and cheaper. The fan gate has no particular advantages over the ordinary mitering gate except the theoretical advantage of automatic operation, and cannot be operated at all if the headwater and tail-water elevation are the same.

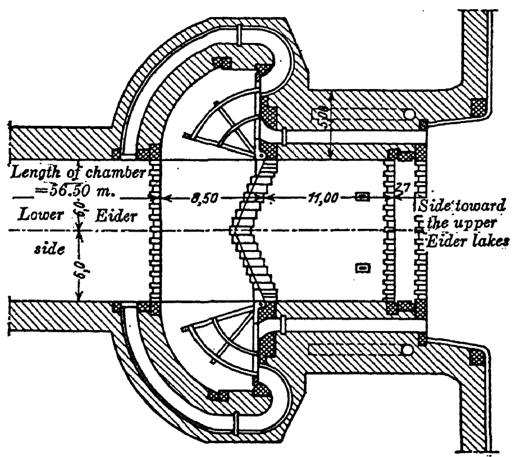


Fig. 435. Fan gate of the Rendsburg locks (an old structure)

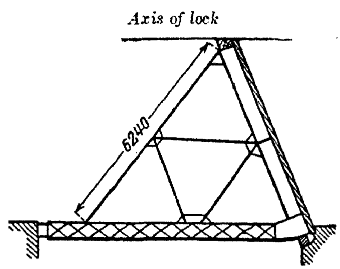
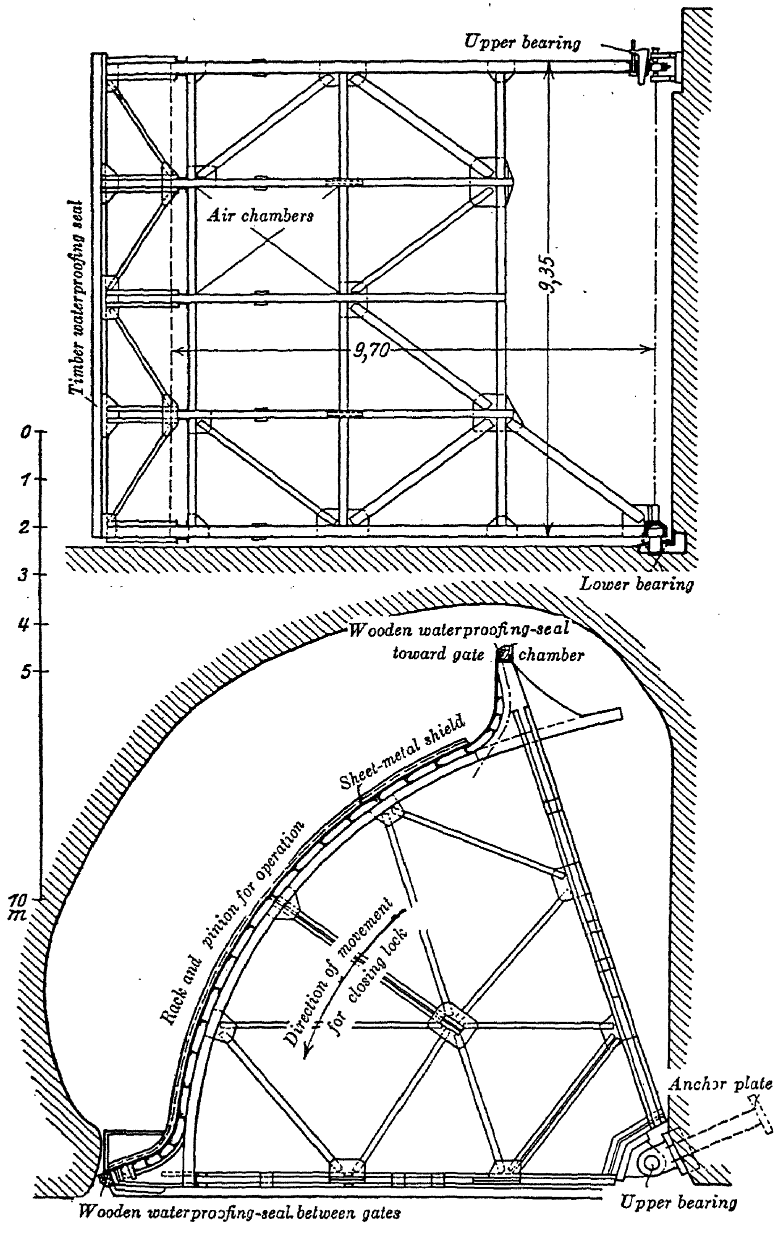


Fig. 436. Fan gate at Rendsburg (new structure)

3. Sector Gates

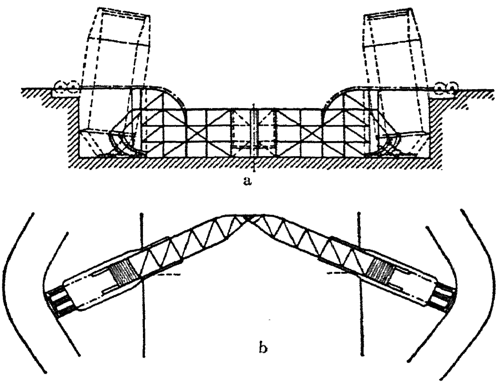
The only sector gates used in locks thus far are those in the Swedish lock, Södertälje. They were used here in order to make it possible to fill and empty the locks through the gates themselves instead of through culverts. The sectors are mounted on vertical axes and turn into hollow chambers similar to the arrangement for fan gates. During the time the gate is in motion, the gate chamber is continually closed from the outer water, so that pressures are exerted upon the cylindrical gate-surface. It is thus possible to open the gate under pressure, the discharge taking place through the slowly enlarging vertical opening between the gates. The arrangement is particularly interesting in that it is the first to allow the lock to empty through the slit between the gates (Figs. 437 a and b).



Figs. 437 a and b. Sector gate at Södertalje, Sweden

4. Bascule Gates

These gates are intended to operate similar to bascule bridges, being rolled out of the water on end (Figs. 438 a and b). They possess the advantage of completely freeing the opening, but require large recesses at the sides. As far as known, no bascule gates have been constructed up to the present time.



Figs. 438 a and b. Bascule gate designed by M.A.N.
a Elevation b Plan

g. Size of Operating Machines

Table 5 gives a tabulation of the performance of a few motors used for operating gates. The computed value of the gate resistance is usually far too small; it is therefore to be recommended that materially larger operating motors be chosen than the computed values would indicate. Care must be taken to allow for sufficient excess head, which requires the greatest portion of the mechanical performance. The size of motor is also dependent upon the amount of time to be allowed for opening the gate. The excess head should be assumed about 10 cm. (4 in.).

H. SLUICES AND VALVES FOR FILLING AND EMPTYING LOCKS

a. General; Classification of Valves; Valve Shafts

According to experiments by Krey, culverts are unnecessary for locks which do not have thrift basins. They may be replaced by the movement of the gates under water pressure. The new gates of the locks at Södertälje, Sweden, are constructed as full sectors with vertical axes and are opened under pressure for filling and emptying the chamber. It was Leonardo da Vinci who recognized that some method must be invented to facilitate the process of opening the gates; this is why he introduced the use of sluice valves in the lock gates, the valves being arranged so that they might be opened under full hydrostatic pressure. It was possible to open them under high pressures because of their relatively small size. In the last analysis every sluice is a diminutive lock gate. It is not improbable that the development of locks will tend toward the elimination of culverts and water will be introduced and

TABLE NO. 5

SIZE OF MOTOR REQUIRED FOR OPENING VARIOUS TYPES OF GATES UNDER VARIOUS CIRCUMSTANCES

Lock	Entrance Width		Height of Gates		Excess Head		Time for Opening	Mechanical Performance in HP for One Leaf	Number of Revolutions
	m.	ft.	m.	ft.	m.	ft.	sec.		n
MITERING GATES									
Old Lock at Münster	10	33	8.5	28.0	—	—	30	5.0	—
New Lock at Münster (Both on Dortmund-Ems Canal)	12	39	10.5	34.4	0.05	0.16	60	10.1	850
Bolzum (Hildesheim)	12	39	11.7	38.3	0.10	0.33	40	13.5	1000
Sault Ste. Marie (United States)	approx. 20	approx. 66	13.6	44.6	—	—	—	50.0	—
Old Lock at Ymuiden	approx. 25	approx. 82	15.0	49.2	—	—	—	41.0	—
TUMBLE GATES									
New Lock at Münster	12	39	4.0	13	0.10	0.33	120	10.1	850
Bolzum	12	39	3.5	12	0.10	0.33	25	13.5	1000

discharged directly through the open gates. The only gates which are applicable to this purpose are either those which may be opened from the top downward or bottom upward or those which are similar to sector mitering gates and allow the water to enter along the axis of the lock. In places where it is desirable to do away with the lock floor and to cheapen the structure by using sheet piling instead of masonry walls, the use of the lock gates for filling and emptying the chamber is an advantageous procedure.

In the case of sea locks, however, it is often impossible to dispense with culverts.

Canal locks which are subject to heavy service, particularly thrift locks, are often advantageously built with bottom culverts. It is possible to arrange an unsymmetrical or even one-sided by-pass system by arranging spur connections below the floor of the lock chamber. Great freedom of design is thereby possible, and often results in a saving in cost. This field for lock valves is bound to remain.

The closures for culverts may be divided into five groups as follows:

Sluice valves,

Segment valves,

Cylinder valves,

Siphon closures, hydraulic closures,

Butterfly valves, register valves, arrangement of Nyholm, Franke, etc.

The design of shafts for valves requires special consideration. With the exception of the low cylinder type of valve, all valves which are used for by-passes require valve shafts. The shaft is vertical and usually rectangular, and is constructed directly in the wall above the culvert. The shaft extends into the culvert as a rectangular well, the oval or otherwise shaped by-pass culvert taking an enlarged rectangular cross-section for a short stretch below the shaft. Grooves in which the valve moves are thus formed. In the case of sluice valves, it is possible to close the shaft above the by-pass in such a manner that the valve is allowed only a very narrow cavity into which it can be raised; however, in many types of valves such an arrangement is not possible. In general, when the valve is closed, headwater rises in the shaft to the same height as that of the water in the head-bay. In river or sea locks the top of the valve shaft must be at a high enough elevation to prevent the water from flowing over it into the lock.

b. Sluice Valves

The oldest known form of valve is the sluice valve. The evolution of sluice valves in locks has been very similar to the development of sluice

gates as movable weirs. In the beginning, simple slide valves were used in small gates under low pressure; today, such valves are used for large sea locks but have developed into sizes of 4 by 7 m. (13 by 23 ft.) and are subjected to heads of as much as 10 m. (33 ft.). Because of the large pressures, the valves are now mounted on roller trains in order to facilitate operating them. The characteristic difference between these valves and the lift gates used in the lower bay of shaft locks concerns the construction but not the sluice plate itself.

Simple slide valves are still to be considered the best for small locks. The operating force is relatively small, operation infrequent, and the power consumption small, so that in many cases in the interest of simplicity it is preferable to use extremely simple and cheap sluice valves, a slightly greater expense then being involved in the operation. In the last analysis the choice lies between adopting a complicated and therefore expensive type of valve (for example, one on rollers) with low-powered operating machinery, or a simple type of slide valve with somewhat more expensive operating machinery.

The simplest form of sluice plate consists of a steel frame made of U-sections, into which wooden plank are fitted. For large areas and high pressures the entire plate must be made of steel.

The simple slide valve moves in steel-lined grooves, the valve being raised and lowered in these grooves. The frame and valve are completely fabricated in the shops and are ready for use in the lock upon being received at the site. Small sluice valves, for example, having a breadth of about 2 m. (about 7 ft.) and a height of 3 m. (10 ft.), may be readily actuated by a single valve-stem. It is preferable to raise the larger sluices by means of two stems. Fig. 439 presents an example of a simple slide valve. Where small locks are not concerned, in which hand operation is desirable provided the amount of transportation is small, only electrical operation should be used. The sluice may be raised by means of a rack, operated by a pinion or by means of a worm and worm gear (Fig. 443). The sliding sluice was used as late as 1894 even for large sea locks such as the Kaiser lock

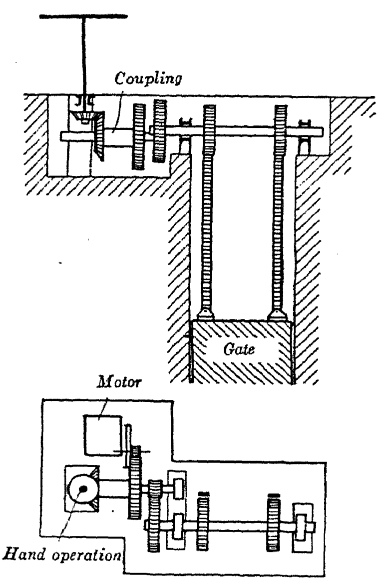
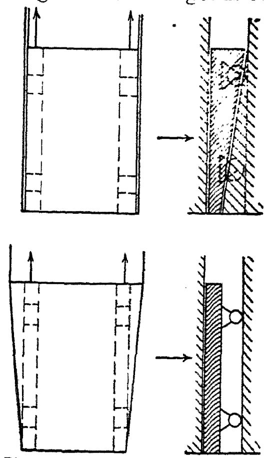


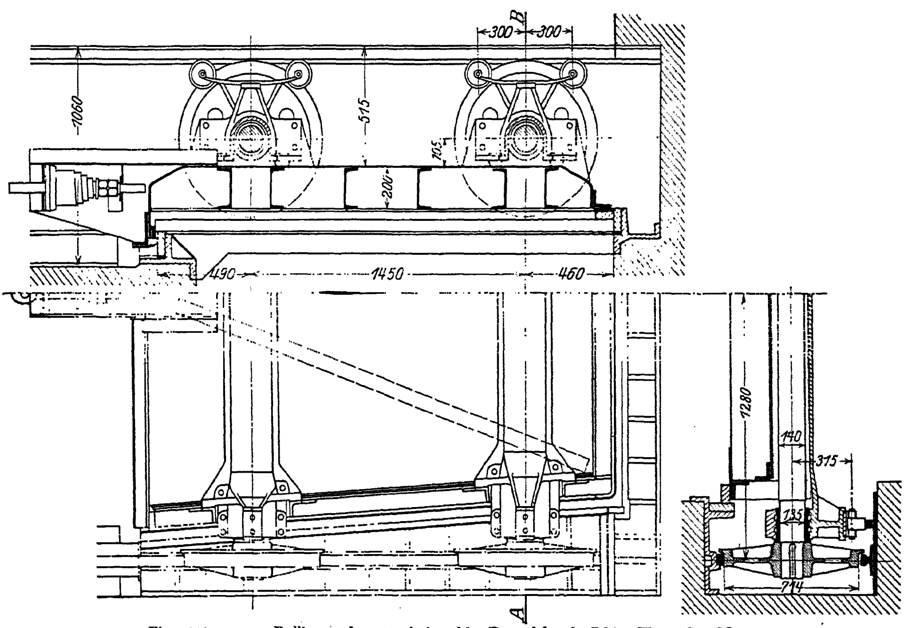
Fig. 439. Sliding gate

at Bremerhaven. The latter possessed a surface area of 2 by 2.5 m. (7 by 8.2 ft.). It is still often maintained, especially by the older engineers, that movable mechanical parts should as far as possible be avoided in hydraulic structures. Although it is fundamentally correct to make the design of underwater structures as simple as possible, with the present status of mechanical engineering it is advantageous to make use of the various innovations if operation is accelerated thereby.

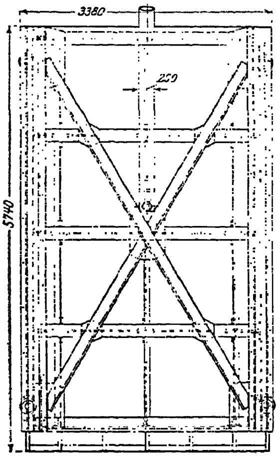
The older types of roller sluices have the disadvantage of not being completely watertight. If the plates are to roll, they must be raised from the sliding plate by means of rollers; the inevitable crevice must be sealed in some manner or other. Spring metal is not always satisfactory for this purpose. A step forward has been made by the invention of the wedge sluice. Two forms have been proposed. The sluice may be either wedge-shaped in cross-section or in elevation (Figs. 440 and 441). Both arrangements make use of a track inclined toward the water-seal so that



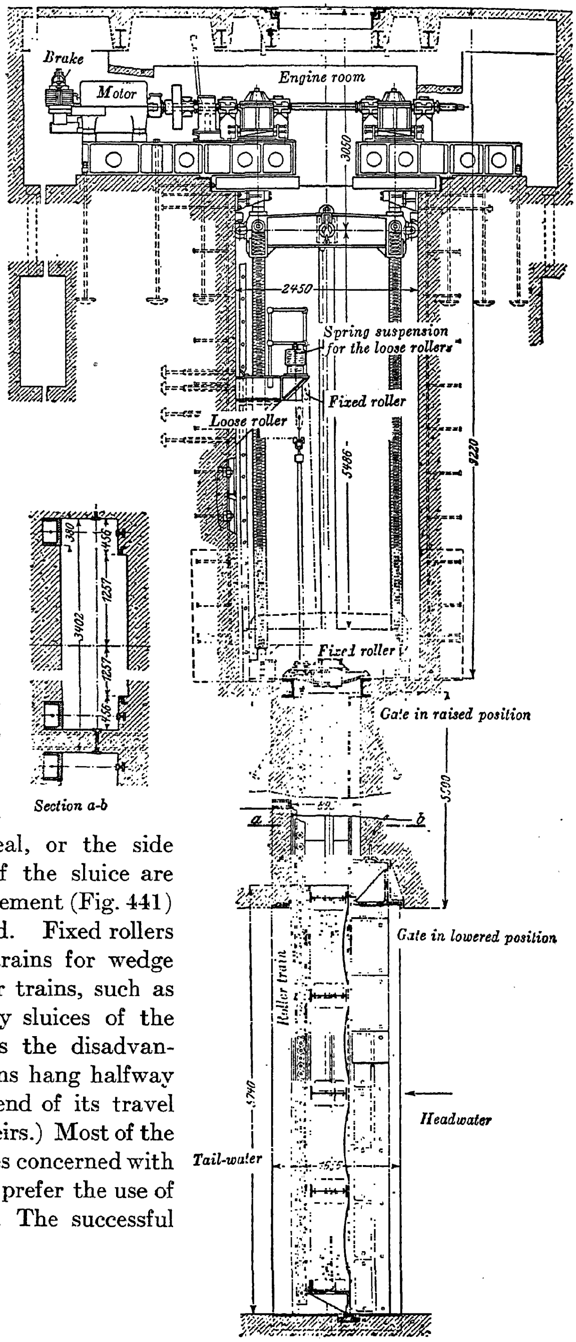
Figs. 440 and 441. Wedge gate forms



Figs. 442 a to c. Rolling wedge gate designed by Freund for the Rhine-Herne Canal lock



Figs. 443 a to c. Gates used for the Panama Canal



as the sluice plate closes, it gradually approaches nearer to the water-seal area until eventually it rests thereon. The sluice reaches the end of its travel the moment it rests on the water-seal. Either the front area is sealed by eventually resting against the water-seal, or the side areas and lower edge of the sluice are sealed. The latter arrangement (Fig. 441) is generally to be preferred. Fixed rollers are preferable to roller trains for wedge sluices. The use of roller trains, such as those used in the Stoney sluices of the Panama Canal, presents the disadvantage that the roller trains hang halfway below the plate at the end of its travel upward. (See Part on Weirs.) Most of the larger German Companies concerned with the construction of locks prefer the use of sluices with fixed rollers. The successful

construction of a sluice with fixed rollers for a water power plant in Sweden in which the sluices are subjected to pressures of as much as 250 tons per sq. m. (51,000 lb. per sq. ft.), on a plate area of 25 sq. m. (270 sq. ft.), is proof that fixed rollers are applicable to all rolling sluices. The arrangement of fixed rollers is so simple as compared to roller trains, that the theoretically larger roller resistance is probably more than counterbalanced.

A sluice plate hung on a strong spring is indicated in Figs. 442 a to c. In this case the water pressure acts from left to right, that is, in the direction B-A, and is continually resisted by the large wheels until the plate disappears into the shaft above. The small spring-connected double counter-rollers keep the sluice from being battered back and forth when not under pressure. The water-seal is effected on the sides and bottom by contact with the wedge-shaped frame; at the top, the water-seal is provided by contact with a strip on the upper section of the frame. A good arrangement is presented by the Panama sluice (Figs. 443 a to c); here the weight of the sluice is balanced by counterpoises. The sluice plates are hung with an elastic connection. Roller trains such as used in this design are not to be recommended.

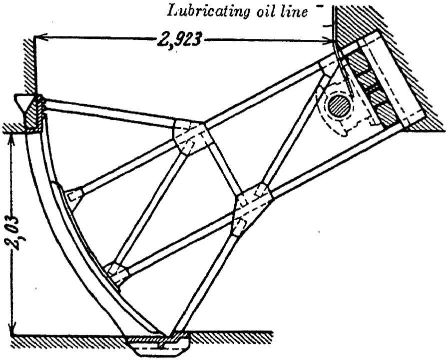


Fig. 444 a. Elevation of the frame

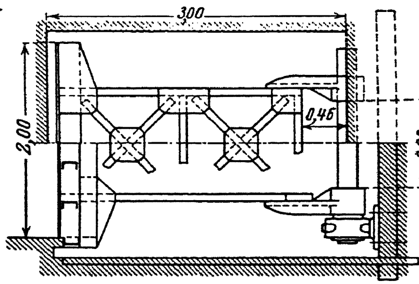


Fig. 444 b. Plan and horizontal section

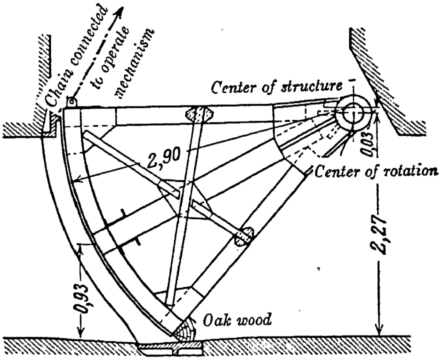


Fig. 444 c. Vertical section through the gate

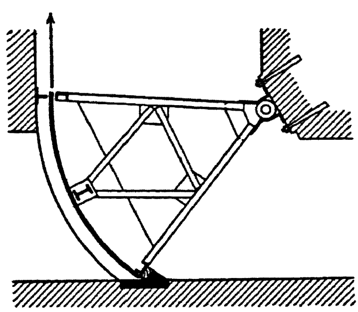


Fig. 445. Segment gate with triangular bracing

Figs. 444 and 445. Segment gates

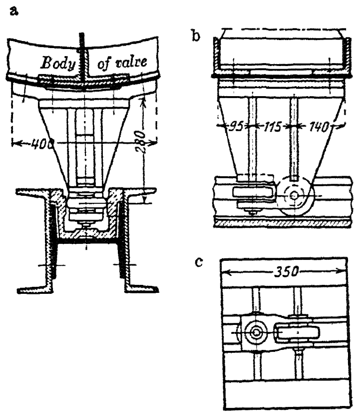
c. Segment Valves

The fundamental difference between a segment valve and a segment weir is that the valve must be provided with a water-seal on all four sides while the weir need be provided with a seal on only three sides, the top being free. Examples of segment valves are presented in Figs. 444 and 445. These segments are each supported at two bearing seats which lie in the shaft in such a manner that the valve can be lifted completely above the top of the culvert. The segment moves in a steel frame. The frame also supports the bearings. The part of the shaft wall to which the bearing seats are fixed must be reinforced by longitudinal steel beams so that concentrated pressure of the bearing is distributed over a broad area. The water-seal is provided at the sides by spring metal; above and below, by simultaneous contact. Particularly good types of construction have been invented by Buchholz.

d. Cylinder Valves

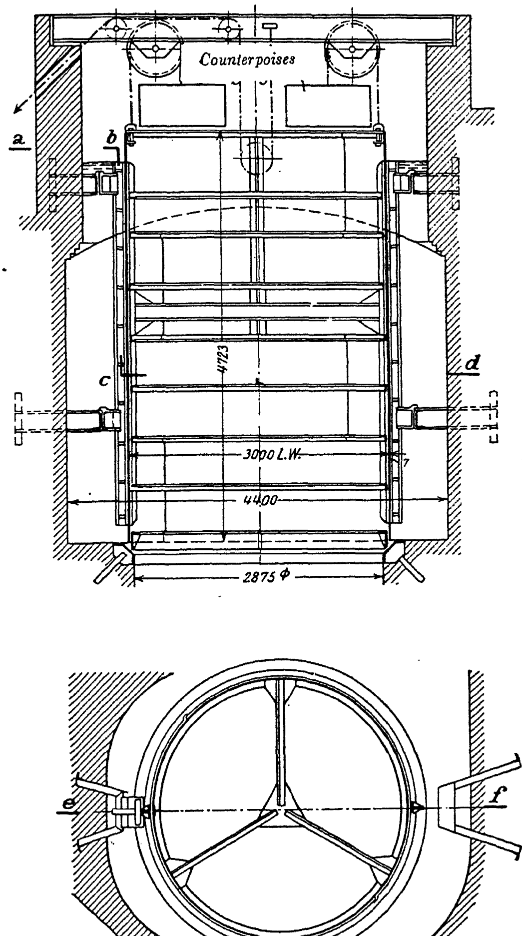
1. High Cylinder Valves (Pipe Sluices)

The high cylinder valve consists of a metal tube which is open at both ends and seats over a drop shaft. (The metal tube is reinforced on the inside by rolled steel shapes.) A closure of this nature is applicable in places where the by-pass may extend downward as a drop shaft from a horizontal plane. One of these types of closure is shown in Figs. 446 a to b. They may be used for thrift basins where they must stand in the open water; however, in ordinary locks they are arranged in a groove in the lock wall. The cylinder is hung from two or three points; the cables connected to these points are carried over sheaves, the opposite ends of the cables being connected to counterpoises. The two-point connection is made more readily than the three-point connection, but the latter is the better. The valve is actuated by an independent cable connected to the center of the cylinder. The cable should preferably be carried over a pulley, and the valve operated by a connection of the cable with an electrically driven windlass. Such an arrangement is shown in Fig. 447. The cylinders are frequently made quite large. For example, heights of 8 m. (26 ft.) and more have been used, and diameters up to 2 m. (7 ft.). A great advantage of this type of valve is that the water pressures are counteracted upon the cylinder itself. If the cylinder does not stand in the open water, the hollowed space in the wall in which it is to operate must be large enough so that the water can flow freely around all sides of the raised cylinder. Eddy currents may develop even in the case of very good designs. It has proved to be advantageous to arrange a vertical strip of sheet metal at the far side of the usually round groove provided



Figs. 447 a to c. Guides for the high cylinder valve of the Niederfinow lock

a Horizontal section b Vertical section
c Plan view



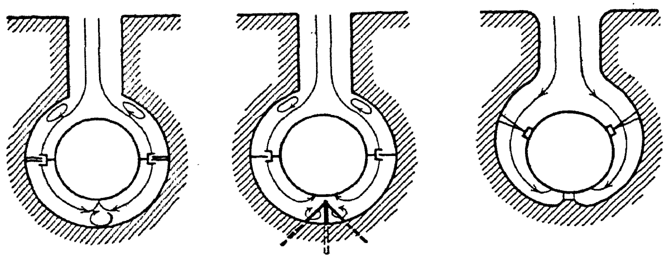
Figs. 446 a and b. High cylinder valve of the head-bay of the lock at Niederfinow

a Section e and f b Cross-section

in the lock wall for the cylinder valve. Such an arrangement hinders the formation of eddies (Figs. 448 a to c). Eddies forming on one side of the cylinder have a tendency to tip it.

Cylinder valves need to be raised only a slight amount for discharging the water. Assuming the radius of the drop shaft to be r, the radius of the cylinder (which must always be larger than the drop shaft) R, the area of the water stream in the drop shaft will be $r^2\pi$. If the cylinder is raised a height h, an area around the circumference of the cylinder of $2R\pi h$ is made available for the entrance of water into the drop shaft. Assuming roughly that $R=r$, it would suffice to raise the cylinder an amount $h=r/2$; thus a cylinder 1.6 m. (5.2 ft.) in diameter would need a vertical travel of only $h=.4$ m. (1.3 ft.) The total lift is usually made somewhat larger than this, but in a case of this nature mak-

ing $h = .5$ m. (1.6 ft.) would suffice. The small amount of vertical travel required by the cylinder does away with the necessity of adding extra height to the superstructure.



Figs. 448 a to c. Development of the cross-section for the cylindrical valves

The cylinder is constructed of sheet iron and has a rounded water-seal at the bottom, which is usually provided with a rubber surface. The drop shaft is usually of "morning-glory" shape. It is important that the cylinder be raised and lowered exactly vertically, because if it deviates from a vertical position, the water-seal will become ineffective.

Open cylinder valves have the disadvantage of allowing large quantities of air to be drawn through the drop shaft with the water. They have, therefore, been provided with a top cover, which also serves to prevent high water from entering the valve from above. The entrance of air diminishes the discharge capacity of the drop shaft and materially decreases the capacity of the valve. The use of a cover to diminish the amount of air drawn into the drop shaft is disadvantageous in that the creation of a partial vacuum within the cylinder results in a downward force of the air pressure from the outside. According to a proposal of Krey, it is preferable to provide a plate within the cylinder supported from the drop shaft in such a manner that the cylinder can move past the plate, a small amount of clearance being allowed between the plate and the cylinder. The amount of air pressure occurring is then very small, and is resisted by the part of the wall below the drop shaft, thereby preventing excess pressure upon the cylinder. The efficiency of the outlet may be greatly improved in this manner.

2. Single and Multiple Cylinder-Valves

The foregoing considerations in which a cylinder is lifted a height of only $r/2$ make it seem practicable to raise a ring having a depth of only about $r/2$ instead of raising the entire cylinder. This thought has actually been put into practice in connection with single and multiple cylinder-valves. In the single valve an arrangement has been devised in

which the cylinder is decreased to the minimum height required to provide the necessary opening for a movable ring (Figs. 449 and 450). For example, if the ring must be lifted a height of 60 cm. (24 in.), the cylinder inclusive of the height of the ring must be somewhat over 120 cm. (47.3 in.) high. The ring lies within the cylinder and is lifted into the fixed bell above. It is unnecessary to make the cylinder

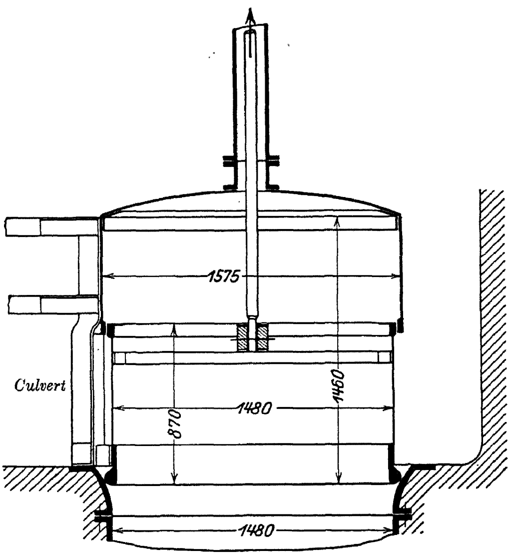
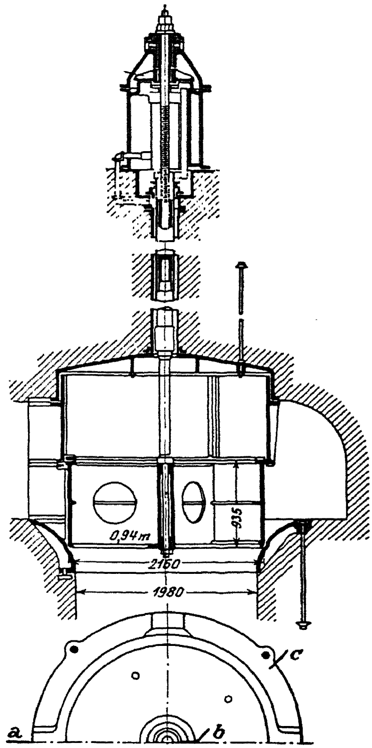


Fig. 449. Low closed cylinder valve of the lock at Brieg in an open shaft



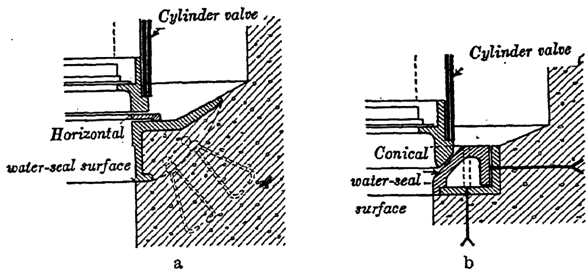
Figs. 450 a and b. Low cylinder valve in the Gatun lock of the Panama Canal
a Sketch a–b–c b Horizontal section

longer than the height of opening inasmuch as the top is closed by a metal cap. A valve of this nature can be constructed as deeply below the water surface as desirable without increasing the cost. The movable ring is hung from a solid or hollow central shaft. The latter projects through a vertical pipe which is concreted into the wall above. The valve may then be counterbalanced and actuated just as the long cylinder valve. A better means of actuation, however, is attainable by the use of a worm gear or rack and pinion.

The movable ring requires a much more accurate water-seal than the movable cylinder inasmuch as the ring must be water-tight at both the upper and lower edges. The ring must seat simultaneously at its lower

and upper edges. Fig. 449 shows a valve of this nature in an open shaft. The hood or bell is supported by girders.

The lower bearing seat may be either horizontal or wedge-shape. In the first case, there is less danger of the valve seating at an angle, while in the latter a closer joint is obtainable. Consequently, the wedge-shaped seat is used almost universally at present (Figs. 451 b to 452), care being taken to guide the ring accurately so as to avoid an inclined position. A particularly good design was devised for the water-seal of the low cylinder-valves of the Panama Canal (Fig. 451). Here the two collar pads lie adjacent to each other so that a rubber surface is provided on both sides. Thus, a water-seal will be provided with excess pressure from either side. It was found that a cast collar may not be cut to a sharp edge, but must be rounded so that the rubber surface will possess sufficient length over which the water pressure may act. Without such pressure, the joint becomes unsatisfactory and the valve will leak.



Figs. 451 a and b. Water-seal surface for cylinder valves
a Horizontal surfaces b Wedge-shaped surfaces

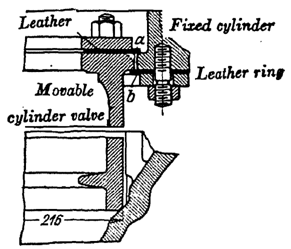
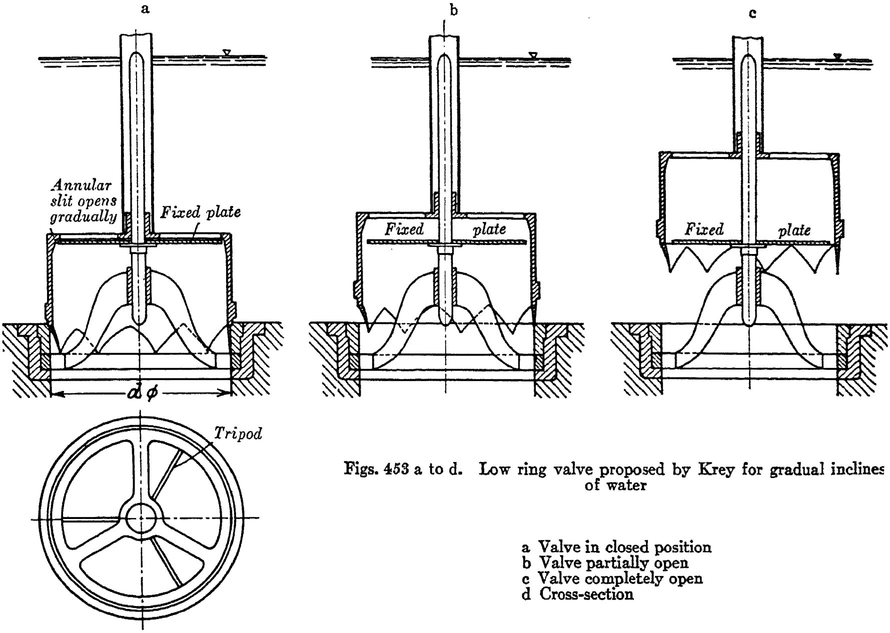


Fig. 452. Water-seal of the lift cylinder for the valve of the Gatun lock

A low cylinder-valve of this nature may be used in a widened section of a horizontal by-pass culvert. The drop shaft then extends from below the valve. A shaft above the valve is usually dispensable; if one is used, a high cylinder-valve might just as well be used also.

The idea of arranging a plate for preventing the entrance of air in connection with high cylinder-valves led to the proposal of eliminating the upper part of the valve and providing a double water-seal for the remaining ring (Figs. 453 a to d). In this case the upper water-seal lies between the ring and the fixed horizontal plate. It would be just as well to anchor the outer hood at the bottom and lift the ring into it so that the low cylinder-valve could be used in the open water, to prevent air from being drawn into the drop shaft by the water. Krey suggests that an improved discharge condition would result from providing a toothed periphery, so that the opening of the valve would take place less abruptly (Fig. 453).

The multiple valve is an outgrowth of the low cylinder-valve. Just as a movable cylindrical ring may be arranged with a fixed cylinder, several rings may be used to operate above each other. This arrangement is used for thrift locks which are described in a subsequent article.



Figs. 453 a to d. Low ring valve proposed by Krey for gradual inclines of water

a Valve in closed position
b Valve partially open
c Valve completely open
d Cross-section

In this case several basins lie over one another. Each basin is separated from the one below by a reinforced concrete floor. A cylinder extends from top to bottom through a series of superimposed basins. The bottom of the cylinder is connected to a by-pass through which the lock is filled and emptied. Each thrift basin (Fig. 455) must be connected to the continuous vertical cylinder in such a way that the basin may be connected or disconnected. For this purpose a multiple cylinder-valve is used, the latter being simply a group of low cylinder-valves over one another. An arrangement of this nature is shown in Fig. 456. Inasmuch as the water flows through the fixed cylinder from various elevations, placing the rings within the cylinder would cause considerable hindrance to movement and might also cause damage as a result of the impact of the falling water upon the inner rings. The rings are therefore put on the outside of the cylinder. Consequently, they can not be suspended from a single central point as in the case of the low cylinder-valve, but must be connected to the outside of the movable ring at two or three points around

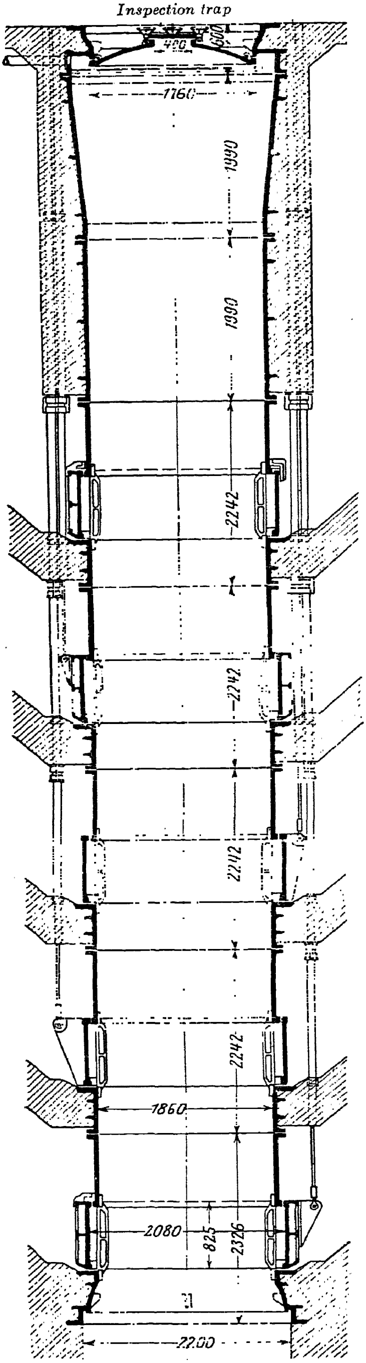


Fig. 454. Multiple pipe valves in the thrift basins of the lock at Anderten (Hanover)

the circumference. Each ring is turned somewhat with respect to the others so that the rods for various

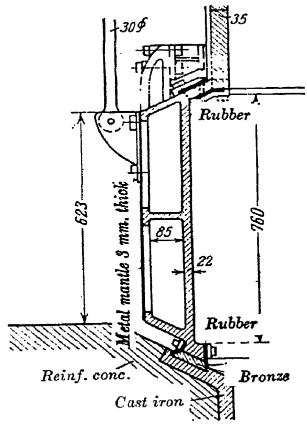


Fig. 455. Cylinder valve of the Minden lock Cross-section and water-seal

rings can lie side by side without disturbing each other. The operating equipment is arranged in a tower above the fixed cylinder. Only one windlass is required, which is

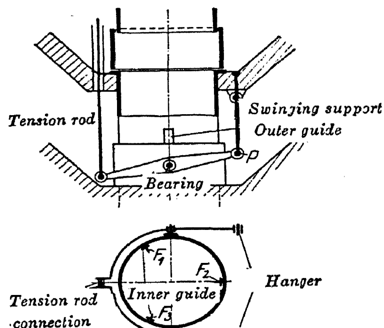


Fig. 456

coupled successively with the various systems of rods. In the case of the shaft lock at Minden, four rings are actuated by a single operating motor.

Fig. 454 shows a multiple ring-valve system originally proposed for the Anderten lock, each ring being hung from three rods. The ring closures are similar to those used in the shaft lock at Minden (Fig. 455) in which a forked operating arrangement was adopted. The figure indicates the location of the necessary outer and inner rubber ring water-seals.

When each of the cylinder rings is hung from three points, a somewhat intricate arrangement occurs at the service motor; for example, in the case of the shaft lock at Minden, 12 lift rods would have to be operated by one motor. Some simplification is introduced by hanging each ring from a movable fork as indicated in Fig. 456. Each ring can then be lifted by one tension rod. This arrangement was used for the recently constructed shaft lock at Anderten. In order to assure vertical movement of the cylinder ring, it is guided at three points, F_1 to F_3, around the circumference of the fixed cylinder. If the turning point P of the fork were fixed in position, the movable cylinder would have to describe an arc about P. Since this is hindered by the guides on the fixed cylinders, point P must be movable transversely. This is accomplished by a swinging support of bearing P. The tension rod extends to above the headwater through a water-tight tube.

e. Siphon Closures; Hydraulic Closures

1. The Hotopp Siphon

There are practically no completely water-tight by-pass closures, although by proper design, the water-seal can be made sufficiently tight to avoid appreciable leakage. An absolutely impervious closure is attainable by a particular type of design for the by-pass. Examples of

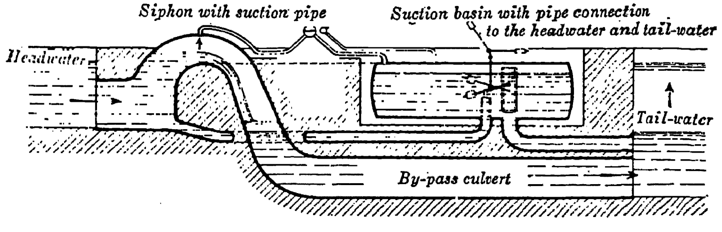


Fig. 457. Hotopp siphon

such by-passes are the Hotopp siphon and the Proetel air-pressure closure. The siphon is not a new device, while the arrangement proposed by Hotopp is essentially new.

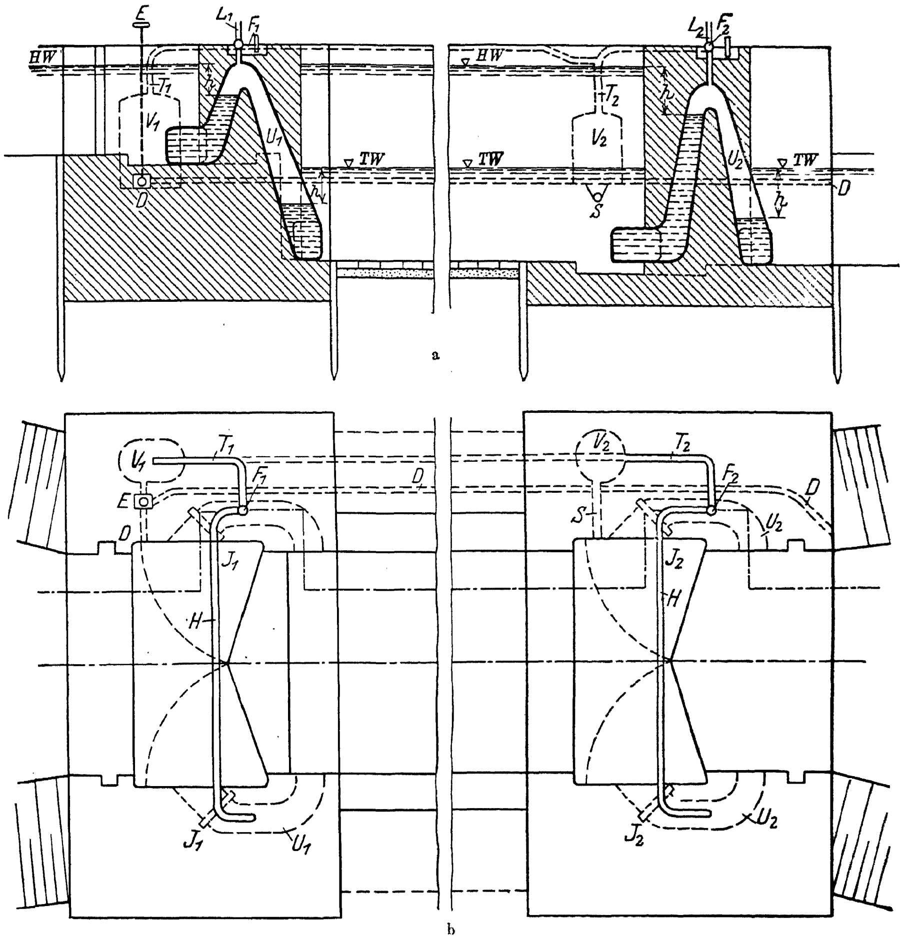
Fig. 457 shows a Hotopp siphon. The lower edge of the culvert between the headwater and tail-water is at one point higher than the headwater level. As the air is sucked from the top of the siphon, water begins to flow over the throat, carrying with it the remainder of the air contained in the siphon, so that within a short time the siphon is completely filled with a stream of water. The air is drawn from the siphon by means of a Hotopp suction chamber consisting of a horizontal steel cylinder built into one of the lock walls, the tank lying between the headwater and tail-water surfaces. It is connected to the tail-water by means of a pipe line. The tank is connected with all of the siphon crests by means of pipes which can be closed off by valves. Since the tank is located near the tail-bay and is connected to the headwater by means of a culvert, the entire difference in head is available. The siphons are put into operation by opening the valves in the pipe lines connected to the crests of the siphons and allowing the water to flow from the tank into the tail-water; this causes the air to be drawn from the siphon and raises the water level in the siphon enough to cause the latter to become primed. The siphon then draws air from the air conduit between the tank and the crest of the siphon so that the tank is again automatically filled by the action of the siphon. This action was not preconceived, but proved to be advantageous. As soon as the tank again becomes filled with water, the connecting conduit between it and the siphon must be closed. The contrivance is then ready for a repetition of the same operation. This ingenious arrangement is applicable to lifts of about 7 m. (23 ft.), but by means of certain revisions may be used for any height of lift. Siphons have the advantage of being completely water-tight, but together with the suction tank are more expensive than other types of closure. It is not necessary to install a suction tank, since an electrically operated suction pump can be used instead.

2. Proetel Type of Air Closure

The design proposed by Proetel is illustrated in Figs. 458 a and b. By-passes U_1 and U_2 are U-shaped so that the crest of the siphon is below the headwater level. Air is blown into this crest in order to keep water from flowing over the siphon. No flow occurs, of course, when the water level within the by-pass is forced to an elevation below the invert of the crest. The air pressure is generated automatically in chambers V_1 and V_2 by water pressure. Chamber V_1, by means of pipe line D and a three-way cock E, can be connected with either the headwater or tail-water; chamber V_2 is connected to the lock chamber by means of pipe S. Pipe T_1 connects the top of V_1 to the crest of by-pass U_2. The crest of each of the by-passes is also provided with an air outlet pipe L, L_1, and

L_2 being also connected with pipe lines T_1 and T_2, respectively, by means of three-way cocks F_1 and F_2.

In case by-pass U_1 is to be closed for the purpose of emptying the lock chamber, the air in V_1 is forced into the crest of the siphon in the head-water bay by opening valve E, chamber V_1 having previously been



Figs. 458 a and b. Proetel air pressure closure

drained and filled with air by allowing water to flow out of conduit D. The air in V_1 is thus compressed and forced into the crest of the siphon through pipe line T_1.

In order to fill the chamber, pipe line L_1 is opened, allowing the air to escape and causing water to flow through the siphon. The air compres-

sion chamber V_1 can again be relieved of the water charge through pipe D during the time that the lock chamber is being filled.

The manner of opening and closing the lower by-pass is the same as in the case of the upper. As the lock chamber is filled, compressed air is developed directly in chamber V_2; thus the lower siphon is closed automatically. Only one cock F_2 is required for closing and allowing the air to be forced from the siphon. In emptying the lock chamber, the aerating pipe must again be closed as soon as flow has started in order to cause the by-pass to act as a siphon. The application of the air closure, however, is not limited by the suction height of the siphon, because the crest of the siphon may be located as far below the headwater level as desired.

If pipes T_1 and T_2 are connected by another pipe line, V_2 can be supplemented by V_1. This is unnecessary, however.

The by-passes on opposite sides of the lock are also operated by air chambers V_1 and V_2, pipe lines H_1 and H_2 connecting the siphons to the air pressure chambers.

This type of closure makes possible a completely water-tight by-pass, and requires no mechanical equipment and no large movable parts. It is lighter and more certain in operation than the siphon closure in which the crest is above the headwater level. Merely a few hand-operated cocks are required, two of these being at the head-bay and one at the tail-bay. The air pressure is provided at a time when there is no flow through the culvert. In case it is desired to be able to interrupt the filling or emptying operation, it is ordinarily necessary either to provide very large air chambers V_1 and V_2 (inasmuch as a great deal of air would be lost due to the action of water carrying the air from the siphon), or to install simple sluice valves J_1 and J_2. The latter would ordinarily remain open. In large locks the air conduits are usually about 12 cm. (4.7 in.) in diameter while pipe line D for filling and emptying the air chamber V_1 is about 25 cm. (10 in.) in diameter. The compressed air might be provided by means of pumps instead of by the air chambers.

f. Older Valve Systems

1. Butterfly Valves

Butterfly valves are arranged to swing on horizontal or vertical axes. The axis (Fig. 459) is so located that the water pressure on one portion is larger than on the other, causing the valve to maintain its position when closed. In this type of valve the water pressure must be counteracted when the valve is to be opened, a condition which almost never occurs in other types of valves. In order to keep this difference in pressures as small as possible, the difference in areas is usually maintained at about 4:5 or 9:10. The valve is then readily opened. In the form

ordinarily used, these valves are not very water-tight, and are consequently seldom used in newer locks. They are swung open and shut by means of a shaft which is propelled either by a gear or, in the case of small installations, by means of a simple lever. A disadvantage in addition to those already mentioned lies in the fact that the valve must either be opened widely or completely closed, because the structure is subject to too large forces when in an intermediate position.

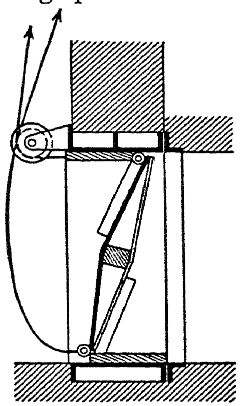


Fig. 459. Butterfly valve with cable propulsion, in the by-pass culvert of a lock

The arrangement has in recent times been used in a horizontal position for the Sault Ste. Marie locks in Canada. Here it was used in conjunction with floor culverts. There is such an excess of water available (flowing from Lake Superior to Lake Huron) that the water-tightness of the valve was not of major importance.

2. Register Valves

The register valve is a variation of the sluice valve. The forces which must be overcome are the same for both, but the time for opening and closing is very much shorter for the register valve. A valve of this nature is illustrated in Fig. 460. The culvert is divided by a transverse grid system which is divided into two sections, equispaced horizontal slits being provided in each section. These sluice plates are arranged in such a manner that one moves upward while the other moves downward, so that when the valve is open the slits are opposite each other. In dividing the discharge area into a large number of small areas instead of one large area, the discharge coefficient μ is greatly reduced, making the total opening area larger than when a single opening is used. In consequence, the pressure or friction to be overcome is much larger for the register gate than for the sluice gate; on the other hand, because of the short travel distance, the operating machinery need not be more powerful for the register gate because the speed of operation may be geared down considerably. There is a certain amount of danger incurred because of the possibility of floating debris becoming caught in the slits and requiring a diver to enter the shaft in order to clear the valve and repair the damages.

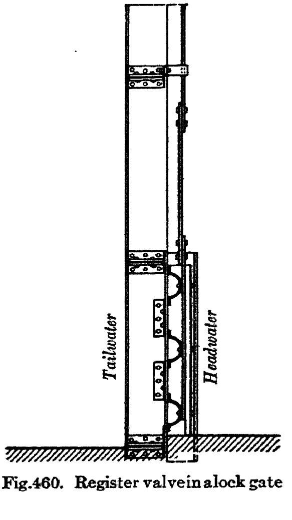


Fig.460. Register valve in a lock gate

This valve possesses a very good point in its favor. In the case of very large by-passes, the sluice plate has sometimes been divided into two plates spaced by an intermediate wall (Panama Canal Locks). It is conceivable that register valves would be successful in such a layout because the travel distance of such a valve would be very small. The register valves in present-day locks are antiquated.

g. Size of Operating Machinery

TABLE NO. 6

SIZE OF OPERATING MACHINERY FOR SEVERAL INSTALLATIONS

Lock	Kind of Closure	Time for Opening sec.	Time for Closing sec.	Motor HP	Number of revolutions per minute
Muenster (n w)	Thrift basin valve 3,000 mm. (117 in.) diameter	180	20	1.8	810
Bolzum	Rolling wedge sluice 3.8 sq. m. (41 sq. ft.); 8.5 m. (27.9 ft.) head	25	25	13.3	1,000
Bolzum	Cylinder valve for by-passes and upper thrift basin; 2,200 mm. (86 in.) diameter; 0.8 m. (2.6 ft.) lift	15	15	8.2	1,000
Bolzum	Cylinder valve for lower thrift basin; 2,200 mm. (86 in.) diameter; 1 m. (3.28 ft.) lift	100	15	8.5	1,000

I. SYSTEMS FOR REDUCING WATER CONSUMPTION

a. General

The necessity of arranging for saving lock water arises when there is difficulty in providing a sufficient supply. The importance of economizing in water consumption manifests itself particularly in connection with summit pools of canals, or in case the water is used to generate power on rivers, and particularly when high-lift locks are used. Contrivances for economizing in lock water are of recent invention. To be sure, it has already been attempted to reduce the water consumption in locks during the past centuries, but no means was found other than adopting low-lift locks. The water consumed does not depend upon the height of the lock lift but upon the depth of the water layer consumed. The old canal and river locks have, for the most part, a lift of about 3 to 4 m. (10 to 13 ft.). Difficulty experienced in building high weirs may have also contributed to the use of low-lift locks. One of the first attempts to reduce the water

consumption in locks by a special process was in an undertaking by Count Caligny in connection with the lock of Aubois on the Loire.

Caligny made use of the kinetic energy of the water in his design. In emptying the lock, the water was allowed to flow through an artificially lengthened culvert, which was so arranged that it could be suddenly closed at the lower end as soon as the water had attained sufficient velocity. This culvert was supplied with a vertical pipe connected to the upper end, the pipe extending to a point above the headwater. When the flow of water was suddenly stopped, the water flowed through the vertical pipe line into an upper basin, lying above the headwater elevation, until the kinetic energy was dissipated. The discharge end of the culvert was then opened until the necessary velocity was obtained and the process repeated and so on until the chamber had been lowered to the tail-water level. The principle is the same as that for the surge tank. Although the scheme proposed by Caligny is very simple, it is not used because it causes irregularity of flow in the lock.

At present the use of thrift chambers is the principal method of reducing the water consumption, although many other proposals have been made for reducing or eliminating water consumption. Among the latter, the methods proposed by Proetel, and the by-pass locks proposed by the author are to be discussed. Earlier designs, such as those of Schnapp and Gerstenberg, Schneider, and others, also present interesting solutions of the problem; but the newer inventions, particularly by Proetel, supersede the former. Probably the simplest solution of the problem consists in raising ships by means of mechanical lifts operated on inclined planes. In the last analysis, these structures are not locks, although they serve the same purpose.

The first fundamental consideration involved in water economy by the use of fixed or movable thrift basins consists in diminishing the pressure head by filling or emptying only a portion of the total lift of the lock. The more completely the available pressure head is utilized, the greater will be the water economy. In movable thrift basins or movable displacement appliances, the pressure difference is reduced to such a small figure that the difference in head between the lock and thrift basin multiplied by the area of the lock chamber gives either the water loss or power required to operate the movable parts (floats or displacers). It also follows that for the same period of lockage the cross-section of the by-pass conduits is a function of the pressure head. If the pressure head H is reduced to h, the by-pass cross-section increases approximately according to the relation $\mu\sqrt{\frac{H}{h}}$ in which μ is a value which may be set equal to unity for small pressure heads, otherwise somewhat less (possibly .9).

The second fundamental consideration is that the longer the route

taken by a drop of water between the chamber and the thrift basin, or the longer the time required to traverse this route, the greater the saving will be.

The action of fixed thrift basins in locks is as follows. In filling the basin the final portion of water is forced into the basin from the headwater; in emptying the lock chamber, this portion of water forces its way into the uppermost thrift basin. When the lock is filled again, this water flows in a layer lying ΔH lower than it was previously; during the second emptying operation, it flows into the thrift basin just below the top one. Thus the water follows a route through a series of thrift basins and finally flows into the tail-water. The longer the zigzag route, the greater will be the amount of saving. In locks applying the by-pass principle, the water flows through all thrift basins. In locks having movable valves (Schneider, Proetel) or displacers (Proetel) the lost or ballast water flows H/h times between the chamber and thrift basins before reaching the tail-water level. The by-pass principle is also applicable to this case. In case no outside water supply is used, mechanical pressure for the movable thrift basins being used instead, the water flows over an infinitely long path for an indefinite time back and forth between the chamber and the movable thrift basins; the water consumption then is theoretically equal to zero.

The volume of the thrift chambers is directly dependent upon the amount of water saved. Locks in which water consumption is eliminated, therefore, require a reservoir capacity (movable as floats or displacers) in which the float or displacer content is equal to the capacity of the lock chamber above the tail-water level.

b. Computation of Water Consumption in Thrift Locks

The arrangement of thrift basins consists in providing a series of basins at different elevations so that when the lock chamber is emptied, the water layers at various elevations in the lock flow into corresponding thrift basins of somewhat lower elevations, the basins thus serving as tail-water reservoirs; in filling the lock, the reverse is true, water flowing from the thrift basins successively into the lock chamber, the basins then serving as headwater reservoirs. Fig. 461 presents the problem involved and its solution. Thrift basins 1 and 2 are provided; the charge I of the lock is allowed to flow into basin 1, and then charge II is allowed to flow into basin 2. Thus, it is evident that no special appliance is required for conducting the water, as thrift basin 1 can no longer supply charge I because the lock and thrift basin will have come into equilibrium. The same is true of II and 2. The remainder flows into the tail-water. If the lock is to be filled, the water from basin 2 is allowed

to flow into the lock chamber until volume IV is filled; thereafter 1 is allowed to flow into space III. Sections II and I must be filled from the headwater. Thus in the complete cycle of operation of the lock, only the content in spaces 1 and 2 was used the second time. This simple example shows that the use of thrift chambers saves only a certain portion of the lock water and that the saving is dependent upon the depth

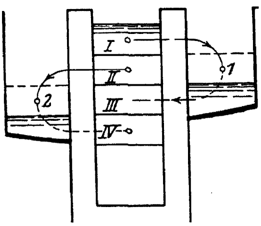


Fig. 461. Explanatory sketch of a thrift lock

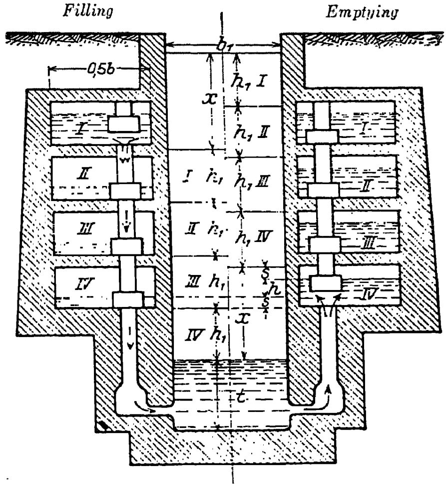


Fig. 462. Thrift lock with four thrift basins Mode of filling and emptying

of the portion which is allowed to flow to the tail-water, this depth decreasing as the number of thrift basins increases. The breadth of the thrift basin is of great importance. The broader the basin, compared to the breadth of the lock chamber, the flatter will be the water layer which lies within it and consequently the greater the saving. An analysis will now be made to indicate the method of determining the amount of the saving (Fig. 462). The right portion of the figure concerns the emptying operation of the lock; the left portion, the filling. The relation of the chamber breadth b_1 to the total thrift basin breadth b (the sum of the breadths upon both sides taken together) is

$$k=\frac{b}{b_1}=\frac{h_1}{h}=\frac{1}{m},$$

in which h_1 and h are the heights of the layers in the chamber and basins, respectively. The equation $b \cdot h = b_1 \cdot h_1$ must be true. There are assumed to be n thrift basins. The layers are not to be completely leveled out, but instead, both during the filling and emptying operation, a small excess head s is to remain between the discharge layer and the surface of the layer received in the thrift basin, and the reverse when the thrift basins are emptied into the lock. The layer which flows off to the tail-water (at the right) is equal to that flowing in from the headwater (at

the left). The height x of these layers is indicated in the figure. Because of complete symmetry both at the top and bottom

$$x = h + s + h_1 + s = (k+1)h + 2s.$$

Furthermore, the difference of the height at one side is $x = H - nh$; if the value h from this equation is substituted into the foregoing equation, and the resulting relation solved for x,

$$x = \frac{(k+1)H + 2ns}{k+n+1}.$$

This value corresponds to the absolute loss in head for a height of lift H. In comparison to the entire charge, the loss V amounts to

$$V = \frac{x}{H} = \frac{k+1+\dfrac{2ns}{H}}{k+n+1}$$

and the saving E will be

$$E = 1 - V = \frac{n - \dfrac{2ns}{H}}{k+n+1}.$$

The most important figure in this connection is that of the loss, inasmuch as it determines the pumping cost of quantity of feed water required. For all practical purposes s may be set equal to zero. In the case of long by-passes, it is not necessary to await complete equalization of the water levels. Caligny showed it to be possible to fill the thrift basins to heights above the successive elevations of the water in the chamber. In the interest of speed in filling and emptying the lock, one usually foregoes making $s = 0$ although the locks are designed according to this assumption. The head loss thus amounts to

$$V_p = \frac{k+1}{k+n+1}.$$

The latter is independent of the height of lift of the lock. The assumption $k = 1$, that is, that the breadth of the lock chamber is equal to the breadth of both thrift basins, is possible only for locks with displaced thrift basins; in this case $V = 2/n+2$. Lock water is generally so expensive that the thrift basins are made broader than the breadth of the lock chamber.

Table 7 is compiled for the loss of water for various numbers of thrift basins and various breadth relations. It is to be noted that the thrift basin increases in breadth as k decreases or m increases. The ratio of the breadth of the thrift basin b_1 to the lock chamber breadth b is given by the ratio $m = 1/k$.

Table 7 shows that in case large numbers of thrift basins are used the water consumed decreases but slightly by adding another thrift basin; for example, if for $m = 2$ the number of thrift basins is increased

TABLE NO. 7

LOCK LOSS V FOR $s = 0$

$\frac{b_1}{b} = m$	$\frac{b}{b_1} = k$	Ratio of water consumption to amount of water necessary to fill lock for $n =$									
		1	2	3	4	5	6	7	8	9	10
		Thrift basins									
3.0	0.33	0.57	0.40	0.31	0.25	0.21	0.18	0.16	0.14	0.13	0.12
2.5	0.4	0.58	0.41	0.32	0.26	0.22	0.19	0.17	0.15	0.13	0.12
2.0	0.5	0.60	0.43	0.33	0.27	0.23	0.20	0.18	0.16	0.14	0.13
1.5	0.67	0.63	0.46	0.36	0.29	0.25	0.22	0.19	0.17	0.16	0.14
1.0	1.0	0.66	0.50	0.40	0.33	0.29	0.25	0.22	0.20	0.18	0.17
0.5	2.0	0.75	0.60	0.50	0.43	0.37	0.32	0.30	0.27	0.25	0.23
0.2	5.0	0.86	0.75	0.67	0.60	0.55	0.50	0.46	0.43	0.40	0.37

from 5 to 6, only 3 per cent of the entire capacity of the lock is gained, although it is to be noted that this 3 per cent of the entire lock charge amounts to $3/23 = 13$ per cent of the amount which must be supplied the lock when 5 basins are used. Thus, if the annual consumption amounts to 100 million cu. m. in case 5 thrift basins were used, a decrease in supply amounting to $100 \cdot 3/23 = 13$ million cu. m. is effected by using 6 thrift basins. Broadening the thrift basins beyond a certain figure results in a still smaller gain. If the thrift basin breadth is increased from 2 to 2.5 times that of the lock chamber, a gain of only 1 per cent of the total volume of the chamber above the tail-water level is obtained. The table also shows that 7 thrift basins having a breadth equal to that of the chamber produces practically the same results as 5 thrift basins which are 2.5 times as broad as the chamber. As the cost of thrift basins is primarily dependent upon their breadths, and since the loading decreases with the number used, a large number of narrow thrift basins is often more economical than a smaller number of broad ones. The cost of valves, of course, plays an important rôle in this connection. Since only one operating machine is required for each multiple cylinder valve, this factor cannot become extremely important, particularly if forked operating mechanism of the nature proposed by Freund is used. The improved form of storage lock invented by Proetel is to be discussed later. An accurate economic investigation is required in any case in order to determine the cheapest form of lock. Fig. 463 presents curves showing relations based upon the data in Table 7.

The structures of this nature which have been erected to the present

time have usually included 4 or 5 thrift chambers having a breadth of 1.5 to 2 times that of the lock chamber. It is possible, however, that increasing the number of thrift basins and decreasing their breadth may be more economical. It is of similar importance to ascertain the error introduced by the assumption that the water level of the thrift basin is the same as that of the lock chamber before the following basin is opened. The difference in height during actual operation is usually fixed at $s=.15$ m. For a total lift H of 15 m. (49 ft.), a corresponding value of V may be determined for various values of k. For $n=5$ thrift basins

$$s=0.15,\ H=15 \text{ m.},\ V=\frac{k+1+\dfrac{2ns}{H}}{k+n+1}=\frac{k+1+0.1}{k+5+1}=\frac{k+1.1}{k+6}.$$

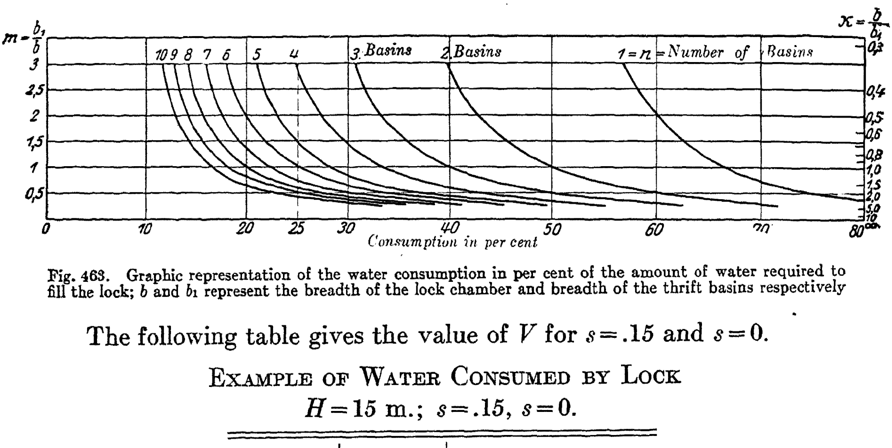


Fig. 463. Graphic representation of the water consumption in per cent of the amount of water required to fill the lock; b and b_1 represent the breadth of the lock chamber and breadth of the thrift basins respectively

The following table gives the value of V for $s=.15$ and $s=0$.

Example of Water Consumed by Lock

$H=15$ m.; $s=.15$, $s=0$.

m	k	V	
		$s=0.15$	$s=0$
3	0.33	0.226	0.210
2	0.5	0.247	0.231
1	1.00	0.300	0.285

This table indicates that the value s unfavorably influences the saving of water, the influence being practically independent of the breadth of the lock and amounting to .016 to .015 H; that is, 6 to 8 per cent of the loss when $s=0$. Thus 108 to 106 million cu. m. would be used annually instead of 100 if $s=.15$ m. instead of $s=0$. Inasmuch as it is possible to eliminate this difference in head entirely (and even cause s to be somewhat less than 0 by the utilization of the kinetic energy of the flowing water), at least part of this loss of water can be eliminated

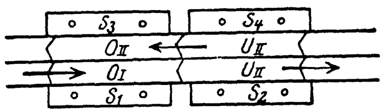
without serious increase in the time required for operation. In the future the value of s should be made equal to 0 or at least a very small value of s should be chosen (for example, $s=5$ cm.).

c. Newer Lock Systems

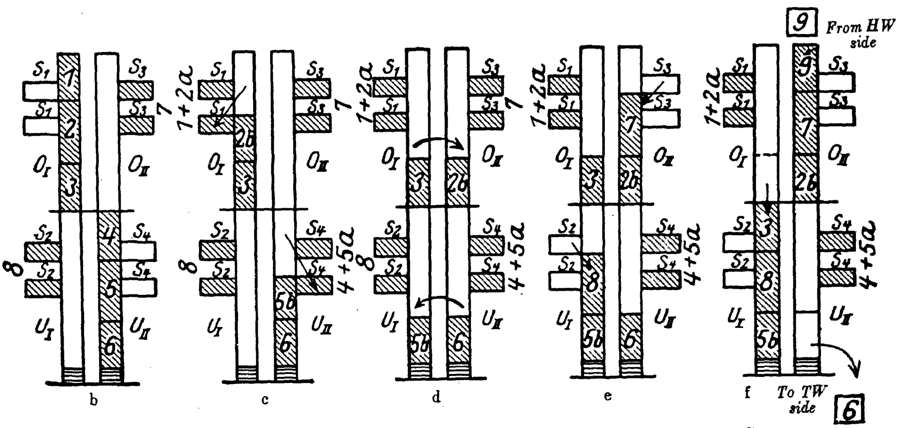
1. By-Pass Flights

In case two two-step lock flights in addition to having longitudinal by-passes are coupled with one another by means of transverse culverts, it is possible to obtain a saving of 50 per cent by equalizing the water levels transversely. The method is accurately described in patent application No. 383093 (Germany). The saving can be increased by adding thrift basins. The mode of operation is indicated schematically in Figs. 464 a to f.

Fig. 464 a represents a ground plan; for the purpose of simplifying the diagram, the movement of the water is indicated as though it took place through simple spillways or siphons instead of through bottom courses. Chamber O I and U II (Fig. 464 b) are considered filled (also



a Plan



Figs. 464 a to f. Double two-stage by-pass flight, *D.R.P. of Siemens Bauunion*, Berlin

thrift basins S_3 and S_2) from O II and U I, while S_1 and S_4 are emptied. The procedure of operation takes place as follows: operation begins (Fig. 464 c) by allowing the water in thrift basins S_1 and S_4 to flow into chambers O I and U II; then (Fig. 464 d) there is an equalization of the

water levels in the parallel chambers. Thereafter (Fig. 464 e), chambers O II and U I are filled from thrift basins S_3 and S_2.

The size and elevation of the thrift basins are so arranged that the water 7 and 8 just suffices for thrift basin S_3 and S_2 (Fig. 464 e) in order to fill the chambers to the lower edge of the upper thrift basin after the equalization of the water levels indicated in Fig. 464 has taken place. This is followed by the filling of O II (Fig. 464 f) by means of portion 9 from the headwater, the discharge of portion 6 from U II to the tail-water, and the discharge of portion 3 from O I into U I. For this purpose the height of the thrift basin is so determined that the remaining water 3 of chamber O I just suffices to supply the quantity lacking in chamber U I (Fig. 464 f); the quantity 9 flowing into O II is equal to that of 6 flowing from U II and also that of portion 3 of step I. The above description thus presents one-half of the procedure of the operation; the condition represented by Fig. 464 f corresponds to that in Fig. 464 b if the chambers were interchanged. The procedure from this point on, therefore, needs no further explanation. The half of the water which did not flow into thrift basin S_1 (Fig. 464 d) flowed from O I to O II. This portion is a part of 2 and is designated by 2_b. During the further progression, it flows toward U II (all within one complete lockage). During the second locking operation the same water 2_b reaches thrift chamber S_4; from here it returns to U II and finally through U I into the tail-water. In this process 2_b has already taken a route through the thrift chamber S_1. By the use of bottom conduits portion 2_b flows through all thrift basins and chambers. The same is true for all water which flows into the upper chambers. The by-pass system makes it possible to use much smaller and higher situated thrift basins than would otherwise be possible with the same quantity of water. In case lockage takes place in only one of the flights and thrift basins are used, it is also possible to use the adjoining flight for the purpose of economizing in water.

It is noted in practice that an equalization as accurate as this is not attainable. Small errors will be introduced which, however, amount to only a few centimeters of height. These can be equalized by connecting the lower chamber to the headwater by means of a conduit of small cross-section operating under a high head. The lower chamber can also be served through the empty upper chamber. It is unnecessary to equalize the levels completely. Operation can take place with partial equalization; flights with thrift basins will then be obtained with small variations in elevation.

The transverse coupling of the flights can be discontinued at any time by merely omitting to operate the transverse valves. Each flight

can operate independently. This case obtains when the amount of transportation is small, when it is not desirable to await a second ship after the lockage of one in a given direction in order to use both flights simultaneously.

It is also possible to use this by-pass principle for flights of unequal size; for example, an upper, comparatively shallow section of the flight might be served by two thrift basins and the lower one by three. The following computations are made for general cases of this nature.

It is considered that the upper lock of the flight provides a lift of H_1; the lower, H_2; the upper is provided with n_1 thrift basins; the lower, n_2. The difference in level between the partially equalized water levels in the chamber is s_k; that between the chamber and the basins, s_b. The relation between the cross-sectional areas of the chamber and that of the basins is $k=F_k/F_b$; as previously $m=1/k$. The following equations are then applicable to unevenly divided flights

$$k=\frac{F_k}{F_b}=\frac{t_b}{t_k} \qquad a=t_b+2s_b.$$

$$H_1=t_k[n_1+2(k+1)]-s_k+4s_b,$$
$$H_2=t_k[n_2+2(k+1)]-s_k+4s_b,$$
$$H_1+H_2=H$$
$$t_k=\frac{H+2s_k-8s_b}{2[.5(n_1+n_2)+2(k+1)]}.$$

The water consumption in terms of the height is

$$v=t_k+a=t_k+t_b+2s_b=t_k(k+1)+2s_b.$$

From this it follows that the water consumption in terms of a fraction of the lift of the flight amounts to

$$V=\frac{i_k(k+1)+2s_b}{H}=\frac{(H+2s_k)(k+1)+2s_b(n_1+n_2)}{2H[.5\ (n_1+n_2)+2(k+1)]}.$$

Choosing $H_1=H_2$, $n_1=n_2=n$, $s_k=0$, and $s_b=0$, it follows that

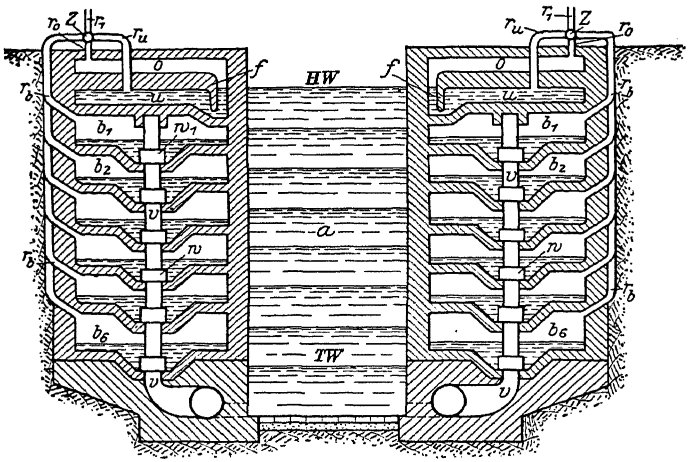
$$t_k'=\frac{H}{2[n+2(k+1)]}$$
$$V'=\frac{(k+1)}{2[n+2(k+1)]}.$$

By means of these equations, tabulated magnitudes of the water consumption for various numbers of thrift basins and various breadth ratios may be compiled. The value n designates the number of thrift basins in one step (not in the entire flight) of such by-pass flights. Even in the

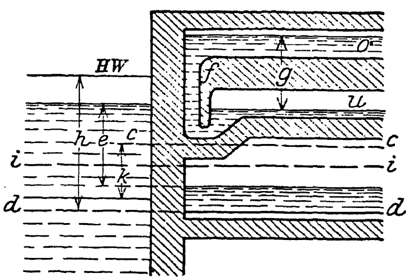
unequally divided form such by-pass flights present great advantages in unfavorable territory, particularly if the height of the lock is so great that it is impracticable to construct a single shaft lock. Lock flights have some disadvantage in being coupled with each other. The disadvantage, however, is very small in the case of a large amount of transportation. If the transportation is small at various times, the coupling may readily be discontinued. Furthermore, during periods of light transportation, one flight can be used as thrift basins for the other. The by-pass principle may also be used for double shaft locks. If it is desired to operate such locks by equalization of the water levels, the thrift basins could be located at higher elevations and made smaller than would be possible for the ordinary shaft locks provided with storage basins.

2. Proetel Storage Locks

A storage lock arrangement proposed by Professor Proetel is indicated in Figs. 465 a and b. In this lock, by the use of fixed thrift basins



a Cross-section through lock and basins



b Details of the completed air chamber

Figs. 465 a and b. Proetel thrift lock

and a partial utilization of the work performed by the water in flowing in and out of the basins, a greater water saving is obtained than in the ordinary storage locks.

Just as in the case of the ordinary thrift locks, basins b are arranged

over one another on both sides of the lock chamber a. Above the top thrift basin b_1 are arranged two other chambers o and u. The basins are connected with the by-passes and lock chamber in the usual manner by valve shafts v which can be closed by cylinder valves w (multiple cylinder valves). All thrift basins and the chambers o and u are made air-tight. The pipe line r_o extends from the roof of chamber o and the pipe r_u from the roof of chamber u; for each thrift basin there are three pipes r_b, r_o, and r_u, which are joined together and can be connected to one another by means of a four-way cock, z, and can also be connected to the pipe line, r_1, which is open to the atmosphere. Chambers o and u are continually connected to each other by means of shaft f.

The above-described arrangement operates in the following manner. When the lock chamber a is full, all basins b are empty; furthermore, chamber u is filled and channel o is empty. The cock z is set in such a manner for thrift basin b_1 that r_b is connected to r_u, and r_o to r_1; then ring valve w_1 is opened. Water flows into thrift basin b_1 and compresses the air contained in the basin, causing the air to enter chamber u through pipe r_b and forcing the water in chamber u into the superimposed chamber o. After pressure equalization (as indicated in Fig. 465 b), the difference in water surface elevation between the lock chamber and the thrift basin b_1, which originally was h, decreases to the amount e as a result of lowering the level in the lock and raising that in the thrift basin. The water level in chamber o during the same time rises an amount g above the level in chamber u. Pipes r_o and r_u now close and r_b is aerated by connection to r_1. The water levels in the lock chamber and thrift basin thereby become equalized and assume the level $i-i$. After this has taken place, r_o is connected with r_b, and r_u with r_1; in consequence of these connections, the water flows out of chamber o back into u and causes a suction drawing air from chamber b_1, raising the water level to $c-c$, while that in the chamber sinks to the level $d-d$. The thrift basin is thus filled to a height k above the water level in the lock chamber.

The remaining thrift basins are filled in a similar manner. The reverse procedure takes place when the lock chamber is filled by a corresponding utilization of the two chambers o and u in which the water is moved back and forth but not consumed. Thus, as is evident from Fig. 542 a, considerable saving in water is attained. If the equalization of water levels is limited to 20 cm. (7.9 in.) excess pressure on one side, in the case of a lock of 20 m. (66 ft.) drop and having thrift basins each twice the breadth of the lock chamber, only one-seventh of the water necessary to fill the chamber is lost. A Proetel storage lock having six thrift basins would probably cost as much as an ordinary storage lock

having eight thrift basins. The ring valves for two thrift basins would be eliminated, but the cost of these is probably counterbalanced by the air conduits which must be installed. Furthermore, in case of the Proetel arrangement, the thrift basins must be air-tight. They need not be absolutely air-tight as the air pressure is quite small and acts for only short intervals. It may be assumed that water-tightness is all that is required for these chambers. Hence, without an accurate cost analysis, it may be assumed that the ordinary thrift basin installation in which eight basins are installed would cost the same as the Proetel system with six thrift basins and two upper chambers.

By equalization to the extent that $s = .2$ m., in the case of a lock having eight thrift basins of width double that of the chamber width, a consumption of 17.5 per cent of the lock capacity would be incurred for a drop of 20 m. (66 ft.). A corresponding consumption by using the Proetel system would amount to 14 per cent. Thus 3.5/17.5 or approximately 20 per cent of the water which would be required for the ordinary thrift lock would be saved by using the Proetel system. For example, if the water consumption for an ordinary thrift lock were 100 million cu. m., a similar layout using the Proetel invention would require approximately 80 million cu. m. annually. This, however, is somewhat offset by the increased time required for filling the chamber. More accurate investigations must still be made before ascertaining the extent of the advantages offered by the new invention. It may be assumed that the advantages are great enough to recommend the new system.

d. The Design of Thrift Locks

As was indicated by the foregoing computations, the elevation of thrift basins must be accurately predetermined. The elevation is determined in such a manner that for assumed values of n, k, and s the value

$$x = \frac{(k+1)H + 2ns}{k+n+1}$$

is fixed and therefrom the value

$$h = \frac{x - 2s}{k+1} \quad \text{and} \quad h_1 = k \cdot h.$$

From this analysis one can immediately determine the divisions of the lock chamber beginning at the top, and also the water surface elevation of the full and empty thrift basin.

Differentiation is made between locks with open thrift basins and

storage thrift locks. In the first, the thrift basins are all adjacent to each other in the form of a flight. They are either arranged in a star formation (such as the Donner Lock of the Elbe-Trave Canal which was designed by Rehder) (Fig. 466), or they are arranged parallel to each other, the uppermost thrift basin being at the head-bay and the lowermost at the tail-bay (Fig. 467). An arrangement of this nature was used for the locks at Niederfinow (Figs. 338 and 339) and Datteln (Henrichenburg). The basins might also be arranged, as proposed by the writer, parallel to the lock in such a manner that long, narrow basins are arranged adjoining each other and extending the full length of the lock (Figs. 468 a and b). The flight might also be arranged as a descending series away from the axis of the lock.

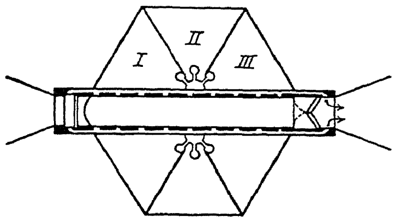


Fig. 466

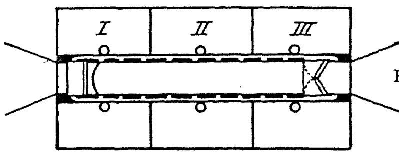
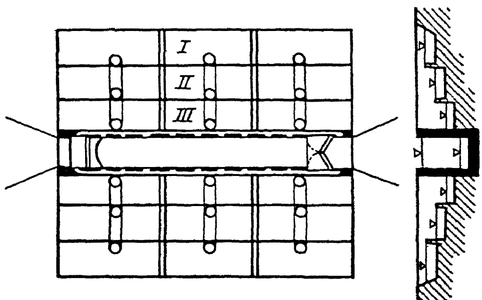


Fig. 467



Figs 468 a and b.

Figs. 466–468. Thrift basin arrangement

The cheapest of these arrangements is probably the intermediate one; but to the present time this arrangement has been accompanied by the formation of undesirable countercurrents in the lock; for example, in the lock at Datteln, such unfavorable flow conditions caused the time required for lockage to be lengthened to twice that which would be otherwise necessary (as is the case of the favorable thrift basins at Minden). In all layouts the arrangement of the inflow and discharge culverts must be such that the water always enters at the same location of the by-pass culverts and also that no countercurrents develop in the by-passes. (The latter circumstance occurs in the Datteln Lock.) The arrangement is very simple in the case of thrift basins which lie over one another so that the multiple cylinder-valve conducts the water from all basins to the same entrance. An arrangement of this nature is desirable also for open basins. This is effected

very readily with three (parallel, longitudinal) basins, since here conducting the water from all points of the basins to the same point of the lock culvert system requires only one transverse culvert for each cylinder-valve.

Individual basins of a system of open thrift basins may be separated either by masonry walls or embankments. If the substrata are not already impervious, the basins should be lined with a loam layer, and the latter paved with concrete slabs in order to prevent erosion by the flowing water. A thin, continuous concrete floor may also be used for the protective cover. The floor is sloped toward the lock and should preferably drain into a drop shaft. The basin must have some capacity below its low water stage in order that the discharge will not be too slow near the end of the basin's discharge period. The floor of the basin at the entrance should lie at least .3 to .5 m. (.98 to 1.6 ft.) deeper than the low-water stage of the basin. The slope toward the entrance may be about 1:50 to 1:100. In the case of open thrift basins it is possible to make the value of m as low as 1; in the case of storage locks with basins over one another, this is not possible because of the necessary space for floors between the basins arranged over one another. An example of thrift locks with open basins is given in Figs. 472 a and b, which presents a design by Plate for the Hansa Canal. Bottom culverts were used in order to get along with thrift basins at one side of the lock. The basins are arranged in a star form to provide uniform flow of the water. Thrift locks with open basins are much cheaper than storage locks, and may serve the same purpose if the layout for conducting the water is well designed. In the case of a large number of thrift basins, the design is not as satisfactory as that of storage locks.

The storage lock system has been used primarily for shaft locks. If the lift of the lock is so great that it exceeds the vertical clearance required for navigation by at least 3 m. (10 ft.) during times of maximum tail-water elevation, it is advantageous to close off the tail-bay by a transverse wall, the bottom edge of which lies above the vertical clearance height required during periods of maximum tail-water elevation; the downstream bay should then be provided with a lift gate which can be supported on all sides (including the top). The lock thus takes the form of a shaft. Locks of this nature are feasible, for example, in the case of 4 m. (13 ft.) vertical clearance, when the headwater lies 5 m. (16 ft.) above the highest tail-water elevation. In this limited case the ordinary mitering gate without an upper bearing is usually cheaper. For shaft locks of 8 m. (26 ft.) height (such as the ascending lock to Lindener Harbor at Hanover), an upper lock closure is sometimes used.

With few exceptions shaft locks are connected to thrift basins, the

latter often being constructed in the open in order to attain a maximum degree of economy. Careful computations must be made in connection with the design of the separating walls. The use of storage basins does not incur as great costs as one might at first anticipate. The lock walls may be made very much more cheaply; consequently, a portion of the cost is saved by the reduction in the cost of the chamber walls. Ordinarily it will suffice to construct the reservoirs (thrift basins) similar to other reservoirs; that is, to support the roof on columns having short drop heads, and sometimes reinforced diagonally. The roofs of the reservoirs must be computed for double flexure. In addition to the bending resulting from the water load from above, a break of a valve may incur hydrostatic uplift against the bottom side of the roof. In order that this hydrostatic uplift may not become excessive, overflows are provided which are connected to a drop shaft, the latter being connected to the tailwater by a special culvert. Thus, for example, if one of the valves in the bottom thrift basin breaks at a time when the lock chamber is

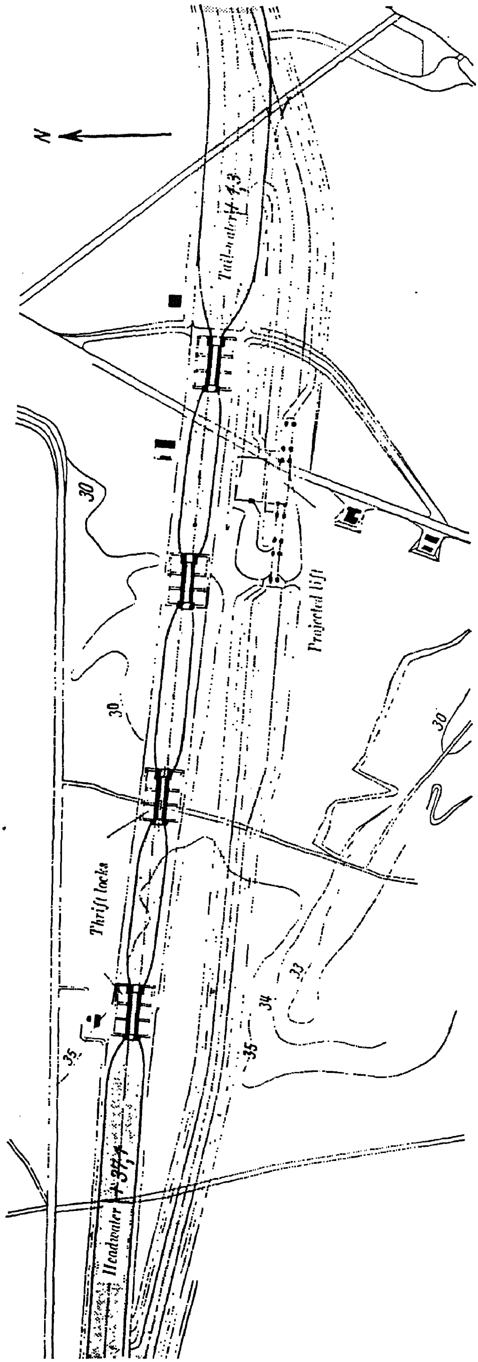
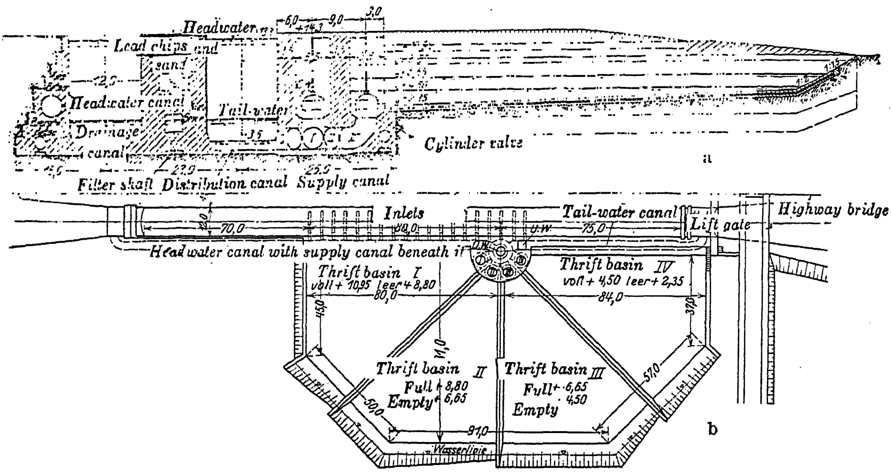


Fig. 469. Lock series with thrift basins at Niederfinow

full (and the basins empty), the water will flow under full pressure head between the headwater and the lower edge of the bottom basin; as soon as the crest of the spillway is reached, water flows from the basin into the tail-water although at an insufficiently high rate. The water level rises until in contact with the ceiling of the basin; the water then



Wasserline = water line

Figs. 470 a and b. Design for a thrift lock on the Hansa Canal

a Section above the thrift basins and section through the thrift basins
b Plan view of one-half of the lock showing arrangement of thrift basins

increases and eventually the full discharge capacity of the spillway is reached. After the water level in the basin once reaches the ceiling, the spillway no longer acts as such but rather as an orifice discharging under a pressure head. The strength required for the roof can be determined by determining the head required to discharge the water from an orifice of fixed size or, conversely, the size of the spillway (orifice) can be so fixed that it will be impossible for the pressure against the roof to exceed a certain figure. In the lock at Anderten it was assumed that the hydrostatic uplift might reach 400 kilograms per sq. m. (82 lbs. per sq. ft.), and the roof was designed accordingly. Because of the possibility of an excess load from either direction, the roofs must be reinforced considerably heavier than would be necessary for loading from one side only. However, they need not be made thicker; the tension steel for loading in one direction, of course, assumes the rôle of compression steel when the slab is loaded from the opposite direction. The occurrence of the hydrostatic uplift against the roof of the thrift chamber is very much out of the ordinary. It seems entirely permissible to design the structure, assuming

working stresses for maximum hydrostatic uplift equal to twice that for normal stresses.

The figures for the locks at Anderten present an excellent example of this type of structure (Figs. 471 a to d). It is to be further pointed out that the water inlet in the thrift basin must be deepened in order to avoid sucking air into the opening. The transverse walls of shaft locks are frequently of very heavy construction. The structure could frequently be made very much lighter by the use of reinforced concrete, as these walls are supported at both top and bottom and can be arranged with intermediate supports.

The route taken by the water in thrift basin locks is of considerable interest. If more than 50% saving is brought about, that is, when more than two thrift basins are used, all of the water from the headwater flows through the thrift basins. When filling the lock, as much water as possible again flows from the thrift basins, after which fresh headwater enters under the layer which flowed from the thrift basins. In emptying the chamber this fresh water from the headwater is forced into the upper thrift basin, etc. If the saving is more than 50%, a portion of the water flows through the thrift basins more than once. The degree of saving and the length of the path taken by the water in the lock are directly related; the law thus holds that the longer the path of the water in the lock, the greater will be the saving. This does not concern the length of the by-passes but the vertical path of the water. Attention is called to the form of the upper inlet of the Anderten Lock. It was designed by the late Krey of Berlin, the form being chosen such that a minimum amount of air is drawn into the lock. The inlet lies on the lock axis in front of the upper gate and then divides into two culverts at the two sides of the lock.

e. Locks Requiring No Water Consumption

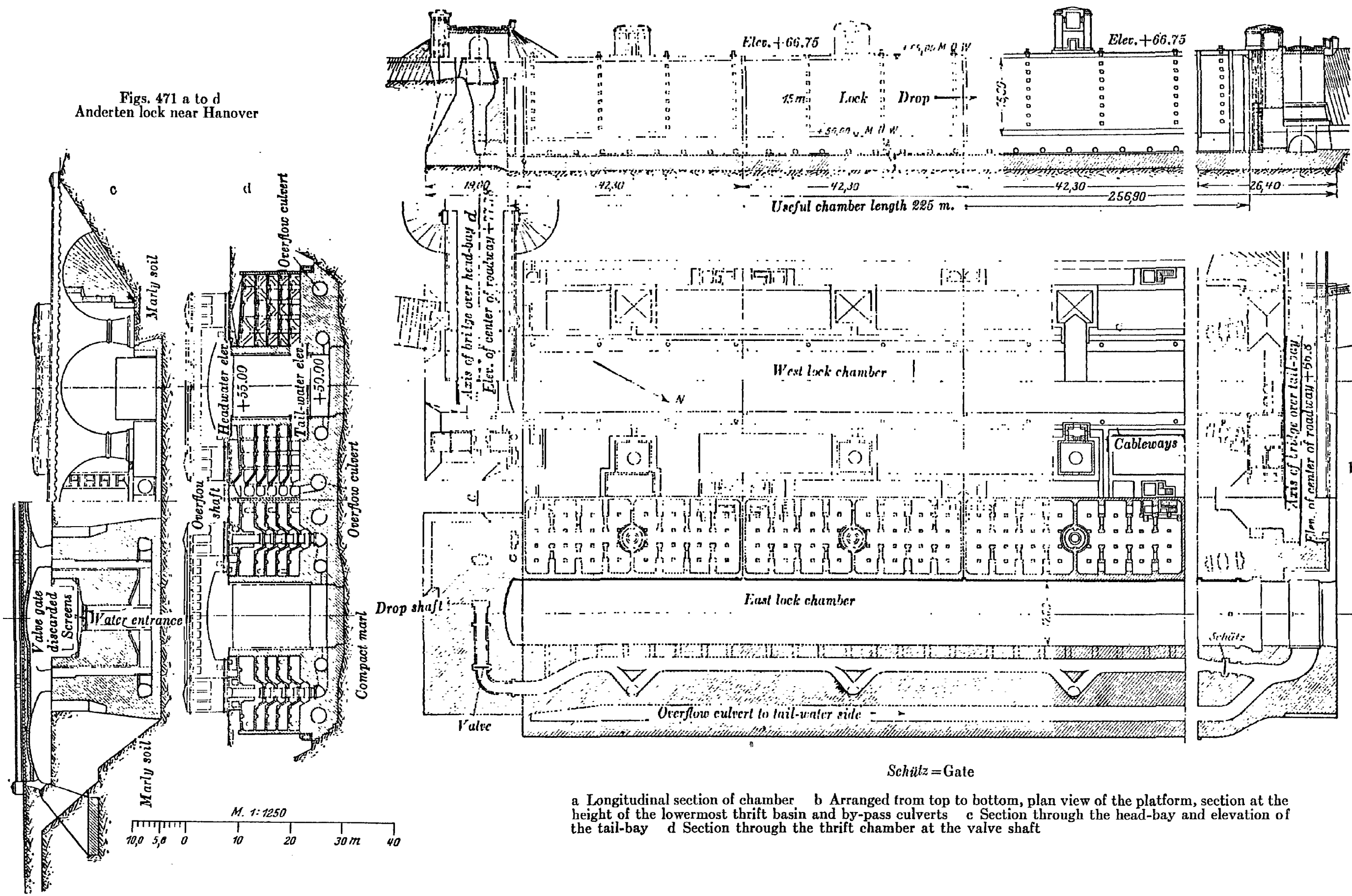
1. Introduction

Various systems have been proposed to eliminate water consumption. The principal difficulties of an economic nature arise from the fact that in all locks in which water consumption is eliminated, a movable container is required which must move upward and downward generally twice as far as the lift of the lock. This difficulty has been considerably reduced by an invention of Proetel. It is highly probable that the Proetel lock is the best design possible for this type.

2. Proetel Displacement Lock

The conservation of water in the Proetel Lock is obtained by the combined action of movable receiving basins and fixed displacers.

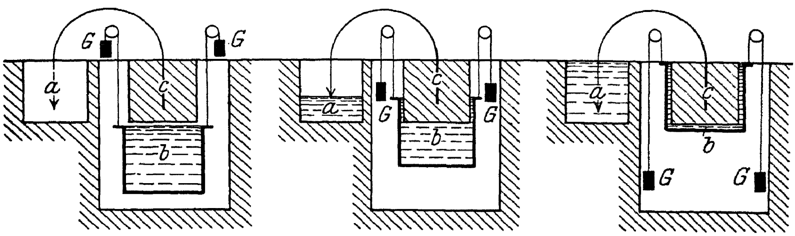
Figs. 471 a to d
Anderten lock near Hanover



Schütz = Gate

a Longitudinal section of chamber b Arranged from top to bottom, plan view of the platform, section at the height of the lowermost thrift basin and by-pass culverts c Section through the head-bay and elevation of the tail-bay d Section through the thrift chamber at the valve shaft

A schematic representation of the operation is indicated in Figs. 472 a to c. In these figures *a* represents the portion of the lock chamber between the headwater and tail-water, *b* the lift basin, and *c* the fixed displacer. The lock chamber *a* is connected to the lift basin *b* by means of a pipe connection. (The latter is not shown in the figure.) The movable receiving basin is hung, using counterweights, racks, float propulsion, or similar devices. The displacer *c* is fixed in position and has a displacement capacity equal to the volume of the lock chamber.



Figs. 472 a to c. Diagrammatic sketch of displacement lock showing beginning, middle, and end position

In Fig. 472 a the basin *b* is in its lowest position and completely filled with water; the lock chamber *a* is empty. After the basin is raised (for example, by increasing the load *G*) the displacer *c* comes into action, displacing a portion of the water. The water level rises and flows through a conduit into the lock chamber. In the middle position (Fig. 472 b), the displacer *c* is halfway submerged and the lock chamber *a* is half full. As a result of the continuous excess weight *G*, the water level in the basin remains continually higher than that in the lock. At the highest position of the basin (Fig. 472 c), the displacer *c* is completely submerged, and the lock chamber *a* completely filled. Inasmuch as the displacer *c* is provided with the same horizontal cross-sectional area as lock chamber *a*, and since both have vertical walls, the same amount of water enters the lock chamber as is displaced by the displacer. The water height in the basin *b* remains constant with reference to the basin, and as it moves freely in the air, the counterpressure downward is the same in all positions throughout its course upward, the water pressure against the bottom of the basin remaining constant.

Equalization by Means of Counterweights

Fig. 473 presents an arrangement in which the weight of the basin *b* is counterbalanced by weights *l*. Counterpoises might also be arranged by hanging weights from cables operated over sheaves, the weight being connected to one end of the cable, the basin to the other (Fig. 472). In order to avoid the use of numerous cables and sheaves and in order to

unite the counterweights into larger units, an arrangement of the type shown in Fig. 473 may be used, counterpoises being fastened to the ends of balance beams *m* and the latter hinged to fulcrum bearing *n*. The opposite ends of the beams are fastened to rods *o* which are connected to basin *b*. The displacer *c* is fixed to girder *i* which is supported on the

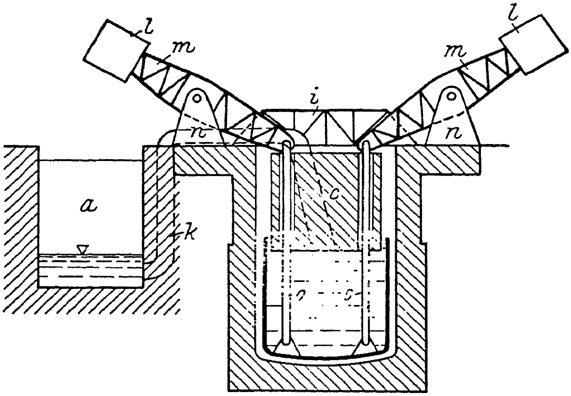


Fig. 473. Arrangement with counterpoise in the case of a single lock

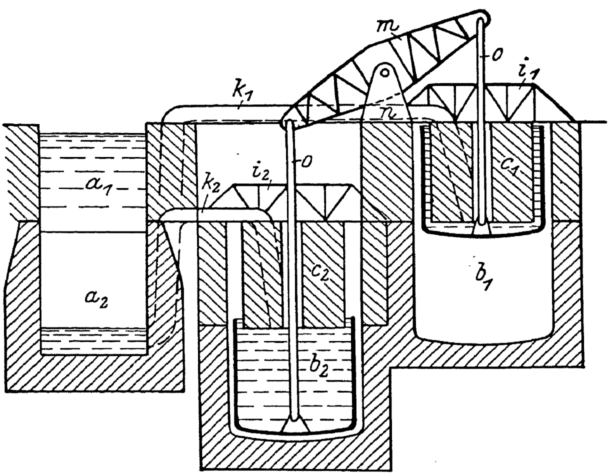


Fig. 474. Arrangement in case of double locks

Figs. 473 and 474. Proetel displacement locks

side walls of the basin pit. If the lift of the lock does not amount to more than about 8 m. (26 ft.), the connection between the lock chamber *a* and basin *b* may be accomplished in a very simple manner by means of a siphon which is designated as *k* in Figs. 473 and 474. The siphon need not be filled each time and then aerated as in the case of the locks. If the

lift is greater than about 8 m. (26 ft.), a movable pipe connection is required; the height of the lift is then unlimited.

The movement of basin b can be accomplished by loading and unloading or by motor power. The counterweights can be replaced by a second lift basin belonging to a second lock (Fig. 474). The second lock may lie either at the same height as the first and form a twin lock with the latter, or it may lie a step higher and form a two-step flight with the first.

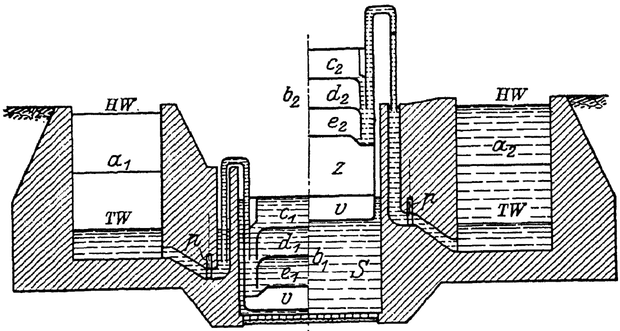
Fig. 474 indicates a lock flight of this nature. Basin b_1 belongs to the upper chamber a_1, b_2 to the lower chamber a_2. Basins b_1 and b_2 which are hung to the balance beam m by means of rods o, equalize each other's weight in all positions. In other respects the arrangement is exactly the same as in Fig. 473. All other features are self-explanatory from the figure. Proetel also made proposals concerning the manner in which the displacer should be designed, indicating that the displacer may be made hollow and provided with a float to which basin b may be hung. The most important factor in connection with locks having displacers is that the arrangement is suitable for a single lock, a single lock flight, or twin locks. It has not been possible to avoid providing a space for the movable basin which must lie deeper than the tail-water level by the amount of the lift of the lock; the basin must have a capacity somewhat greater than the volume required for filling the lock from the tail-water to the headwater level. If the chamber has inside dimensions of 100 by 12 by 8 m. (100 by 39 by 26 ft.), the basin must have a capacity of approximately 10,000 cu. m. (354,000 cu. ft.).

3. Proetel Float Locks Operated by Compressed Air

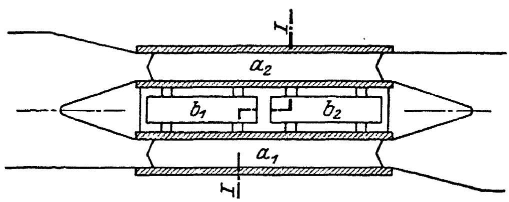
The lock may be designed either as a twin lock as indicated in Fig. 475, or as a single lock. In the first case a diving pit is arranged between lock chambers a_1 and a_2; this pit contains the floating lift basins b_1 and b_2 which may be raised and lowered. The two basins serve to raise the water flowing from the lock chamber. They each contain three similar chambers, c_1, d_1, e_1 and c_2, d_2, e_2, respectively, and also a float chamber v for equalizing the dead load. The chambers c_2, d_2, e_2 move within a zone which extends above chambers c_1, d_1, e_1, a distance half as high as the lift of the lock; inasmuch as the lift basin b_2 similarly to b_1, lowers into the diving pit, a correspondingly higher displacer z is arranged between e_2 and v.

Each of the receiving basins has a combined capacity within its chambers c, d, e, which is equal to half the quantity of water required to fill the lock. In order to maintain equilibrium in all positions, it must be emptied in the highest position and filled in the deepest position; all

three chambers must be filled at uniform rates upon sinking and emptied at uniform rates upon rising. This is brought about by aerating only the middle chambers d_1 and d_2, while c_1 and c_2 have rarefied air within them, and e_1 and e_2 are filled with compressed air. If the vacuum in chambers c, on the one hand, and the excess pressure in chambers e, on the other, just correspond to a water column equal to the height of the



a



b

Figs. 475 a and b. Proetel float locks with compressed air operation

a Cross-section through the double lock and float chamber
b Plan view of a double lock

basin chambers, and if the air pressures remain constant, the water in all three chambers will be at the same depth. The constancy of the air pressure is conditioned by means of a pipe extending from the top of chamber c_1 to the top of chamber c_2, and similarly a pipe from the top of chamber e_1 to the top of chamber e_2. (The two pipe lines are not indicated in Fig. 475 a.) Thus the air passes from c_1 to c_2, and similarly from e_1 to e_2, upon filling one of the basins and emptying the other. Since the same amount of water enters c_2 and e_2 as flows out of c_1 and e_1, the amount of space occupied by the rarefied and compressed air remains constant; therefore, the air pressures remain unchanged.

The open basin chambers d_1 and d_2 have a communicating connection with lock chambers a_1 and a_2, the connections consisting of siphons extending into open shafts. By means of special appliances (which are

not described here), a connection of this sort can be made independent of the suction height of the siphon. The lift basins have siphons of this nature on both sides connecting them to the lock chambers. Gate valves p are constructed into the connecting culverts. The air conduits can also be made completely tight without sliding connections.

The manner of operation is as follows: If, as indicated in Fig. 475, lock chamber a_1 is empty and a_2 full, receiving basin b_1 will be in its lowest position and b_2 in its highest position. By a proper setting of the valve p, the basin b_1 will be in communication with lock chamber a_1, and b_2 with lock chamber a_2. Now if b_2 is lowered by means of mechanical power or an applied load, the water rises in float space S and lifts b_1. Basins b_1 and b_2 are hydraulically coupled through the common float space S. In thus lifting b_1 and lowering b_2, b_1 empties into the lock chamber a_1 and b_2 is filled from a_2. At the end of the basin movement, lock chamber a_1 will have been half filled and a_2 will have been halfway emptied. Then by changing the position of valve p, basin b_1 will be connected to a_2 and b_2 to a_1. After b_2 has again been raised and b_1 again lowered, a_1 will be entirely full and a_2 entirely empty. The same procedure takes place in emptying a_1 and filling a_2. The magnitude of the constant load upon basin b_2 corresponds to the constant pressure head s between b_2 and a_2 and between a_1 and b_1. It is of no consequence whether the pressure is brought about by means of weight (water ballast) or applied by a motor. The elevations of the floats are fixed by the aerated chambers d_1 and d_2. In the beginning of the filling a_1 the water surface of d_1 lies a small distance (s) higher than the tail-water level of a_2. With the reversal of the float movement the bottom of d_1 lies at a small distance (s) below the MW, the water surface of d_2 a short distance (s) above the MW of the (half filled) lock.

By the use of three chambers in the lift basins, a total lift of 20 m. (66 ft.) can be obtained. In the case of low lifts, the pressure chamber may be dispensed with; for very high lifts, two pressure chambers may be used. The lift basins may not have less than two chambers. It is to be recommended always to design one of the chambers as a suction chamber in order to reduce the depth of the diving pit. Since the suction chambers are the upper chambers, the water will be drawn to a height above the water level of the lock when the thrift basin is in its lowered position. The depth of the float space of the thrift basin will be reduced by this amount. This system is probably superior to the displacer system for twin locks.

CONCLUSIONS. The Proetel locks without water consumption are the most comprehensive that have been thus far invented. The earlier inventions of Schnapp-Gerstenberg and Schneider, respectively, were

considerably more complicated and also more expensive. Proetel's invention forms the logical progression from these earlier designs, although he solved the problem in a different manner. His principal invention was made 20 years ago. The important feature lies in the fact that it is now possible by means of movable chambers, having a capacity of half that of the lock chamber, to effect a saving of all water required. Nevertheless, in connection with this layout, either a twin lock is necessary or lift basins must be used having a capacity equal to that of the lock. The previously described displacement locks without water consumption are still more expensive. It is probably that the Proetel storage locks are superior to the Proetel locks without water consumption. Absolute judgment in this connection is possible only after making accurate comparative estimates. The patents of Proetel locks belong to the Siemens Bau-Union of Berlin.

J. INCLINED PLANES AND MECHANICAL LIFTS

a. Inclined Planes

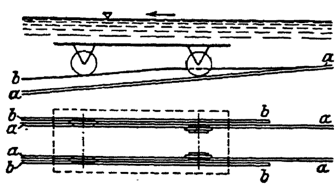
1. General

As was already indicated in the introduction, inclined planes were used for transporting ships from one water level to another before locks were invented. Nowadays, because of the very great differences of water level which must be surmounted, the same solution is used but in a newer form. There are a great many engineers that advocate the use of inclined planes for the purpose of raising and lowering ships to different water surface elevations. There are two types of inclined planes and two methods of moving ships, which are to be distinguished from each other. Both longitudinally inclined and transversely inclined planes are used. Ships may be moved along the inclined plane either by means of a water chamber (that is, moving the ship as it floats) or on more or less flexible supports (that is, moving the ship dry). Dry transportation was last used in connection with the inclined planes of the Elbing-Oberländer Canal. The inclined planes of this canal have more recently been replaced by shaft locks. Dry transportation in this instance did not prove entirely unsatisfactory, although a certain amount of damage to the ships was attributed to this procedure. A ship undergoes deformations which are dependent upon the wave action and nature of the loading; however, all such deformations taking place as a result of water pressures are of a very elastic nature. On the other hand, the deformations incurred in moving ships on a rigid dry plate are inelastic and persistent as compared to the deformations incurred as a result of water pressures. Riedler of Berlin therefore proposed supporting the barges on a bed of hydraulic

shores pressed against the bottom of the ship while it is floating. The inlet culverts would then be closed so that all shores would have to remain in the fixed position, causing the ship to remain in the same relative position which it had while in the water. No structures of this nature have thus far been erected, but unquestionably this method and similar methods are applicable to overcoming the disadvantages resulting in the dry moving of ships.

2. Longitudinally Inclined Planes

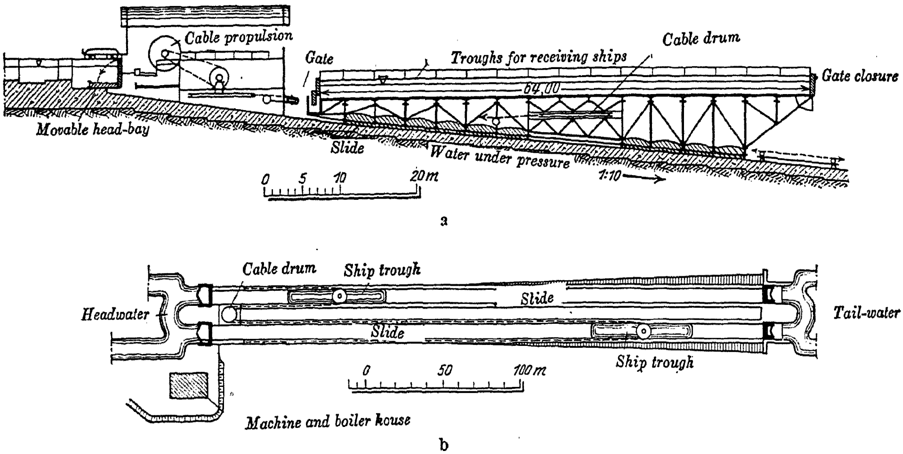
A discarded form of the dry method is indicated by Figs. 476 a and b. Different rails are used for the front and rear wheels of the towing cart; the rails being displaced from each other horizontally an amount equal to the distance between the front and rear wheel axes. Along the lower part of the track the front and rear wheels roll on parallel rails in such a manner that the top of the cart meets the bottom of the ship in a horizontal position; the rear wheels then roll over a horizontal stretch of track while the front wheels continue up the incline, causing the cart to take an inclined position for the remainder of the journey up the plane. After going over the divide into the upper pool, the cart again takes a horizontal position so that the ship is lowered into the upper pool in a horizontal position. Inclined planes of this nature used on the Elbing Canal raised ships as much as 25 m. (82 ft.) over an incline of 1:12. Heights as great as this may now be surmounted by the use of shaft locks where foundation conditions are satisfactory; in any case, two-step lock flights could be used.



Figs. 476 a and b. Diagrammatic sketch of arrangement for dry transportation over an inclined plane. Upper figure shows section, lower figure plan view

Another type of longitudinally inclined plane is shown in Fig. 477. In this layout the ships are moved wet; that is, in a floating position. A water chamber is provided for receiving the ship and transporting it on wheels which move over a steel scaffolding. The plane is intended to be provided with two tracks so that one chamber is at the top while the other is at the bottom, providing a weight equalization of the transporting cars. A number of cables are fixed to each chamber, the latter moving over rollers at the upper end, the two ends of each cable being fastened to corresponding points on the two cars. The cars may be operated by direct propulsion of the connecting cables, or a special tension cable may be used independent of the counterpoise cables. The operating cable must merely overcome the rolling frictional resistance which theoretically amounts to about .01 of the load but actually may reach three

times this amount. Haniel and Lueg have proposed replacing the wheels by sliding shoes which are to be lifted by hydraulic pressure. Each shoe is to consist of an open steel chamber which is to be waterproofed around the bottom edge by means of a leather brim. It is intended to provide so great a hydraulic pressure in these chambers that water continually seeps under the leather brims, the pressure being



Figs. 477 a and b. Inclined plane proposed by Nakonz

a Section through the trough and upper pool b Plan view

generated from above. Thus the sliding shoe does not slide upon the leather brim but upon the compressed water which seeps under the leather brim. According to experiments, the frictional resistance for such a layout would amount to only 1/112 to 1/300 of the weight. During freezing weather glycerine should be added to the compressed water. This proposal is theoretically very interesting; but, it proves impracticable, if not impossible, to place sliding shoes in such a way that the leather brim will slide with a filament of water under it at all points along its route. The wheel systems would seem to be the most reliable. The water trough must be closed at both ends by movable gates, and corresponding gates must be arranged at the ends of the canal pools. All gates must be closed while the cars are in motion. To provide transition of a ship from or to the canal, both gates must be opened at the point of contact. For this purpose, it is preferable to have the two gates coupled together so that they may be lifted and lowered by the same apparatus. Lift gates are used because they require the least amount of space. The arrangement of the gates is the same as that used for troughs in connection with vertical mechanical lifts.

The arrangement as described has the inherent disadvantage that the two troughs are dependent on each other for operation. This disadvantage may be overcome by using a correspondingly loaded car as the counterweight. The power consumed will be increased thereby, inasmuch as the same power is required as would be necessary for moving only one ship. Theodor Hoech proposes allowing the counterweight car to operate below the trough on the same track used by the latter so that only one inclined plane is required.

The longitudinally inclined plane possesses the advantage that a ship moves forward in its direction of travel at a velocity even greater than that at which it progresses along the canal. It is particularly applicable to high lifts. In the case of lifts greater than 50 m. (164 ft.), only longitudinally inclined planes should be used.

3. Transversely Inclined Planes

Transversely inclined planes do not possess the advantage of transporting barges in their direction of travel. Nevertheless, they possess the advantage of a cheaper substructure for the trough. In the case of a steep, longitudinal plane, the trough must be equipped with a corresponding length of substructure at the lower end. For the transversely inclined plane this substructure is dependent only upon the narrow

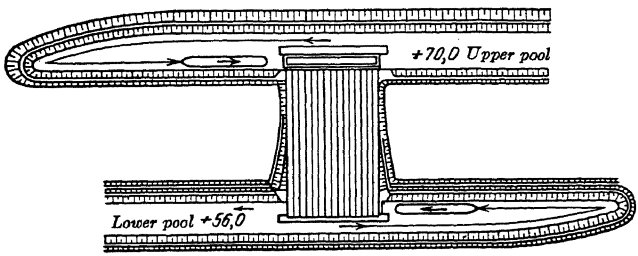


Fig. 478. Inclined ship lift proposed by Th. Hoech

breadth of the trough. A transversely inclined plane of this nature was constructed at Foxton, Grand Junction Canal, England. Here a double plane was constructed in which the two troughs are completely independent of each other.

A transversely inclined plane design is shown in Fig. 481. Here the trough must be closed at both ends by gates, while only the upper pool requires a closure at one end. No gates are required in the lower pool, if the trough is lowered into the tail-water to the extent that the water level in the trough comes to the level of the tail-water surface. Large differences in the elevation of the headwater can very readily be equalized in this manner because in the case of high headwater the trough need

only be lowered into the tail-water a correspondingly greater amount in order to bring the water level of the trough down to that of the tail-water. However, in the case of ships of large dimensions [3,000 cu. m. (about 3,300 tons) displacement or more], strong currents cannot be avoided, as a large quantity of water must be displaced within a short time. It is therefore preferable to make the connection with bays as previously described, thus requiring closures for the trough and bays by means of gates.

The trough must be cut at an angle in plan at both ends so that it will form a good waterproof connection with the bays, respectively, of the upper and lower pools (Fig. 479). The transverse frames of the trough are then automatically placed against the ends of the bays. An arrangement very similar to this was used for the lift of Henrichenburg.

A good arrangement for transversely inclined planes was suggested by Theodor Hoech. Fig. 478 shows the plan view of the improved form. Hoech proposes to allow the trough to be transported into a bay so that it is accessible from both sides. It is intended to have the trough dip into the tail-water. In the upper pool, all four gates are to open simultaneously so that the rising ship can leave the trough by moving out forwards, and the ship which is to be lowered can be pulled into the trough backwards. One ship might enter the trough and another one leave it simultaneously with an arrangement of this nature. In the tail-water, the movement of the ship takes place in the reverse direction from that in the headwater. An appreciable waiting period is incurred only by the entering ships, as they must travel around the outside of the trough and then be towed backward into it. Hoech indicated a very short piece of canal in back of the trough; it is preferable to make this extension fairly long in order to allow plenty of room for waiting traffic. The inclined plane proposed by Hoech has practically twice the capacity of an ordinary structure of this nature. He also proposed arranging counterpoise cars loaded with brick, these cars to operate below the trough.

In case of very great differences in elevation between the headwater and tail-water, the inclined plane can be made into a short railroad as proposed for the moving of sea ships. (An arrangement of this sort was proposed in place of the Panama Canal.)

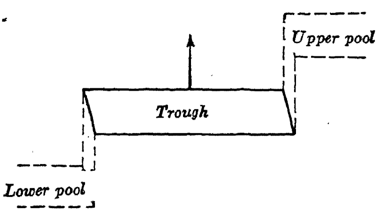
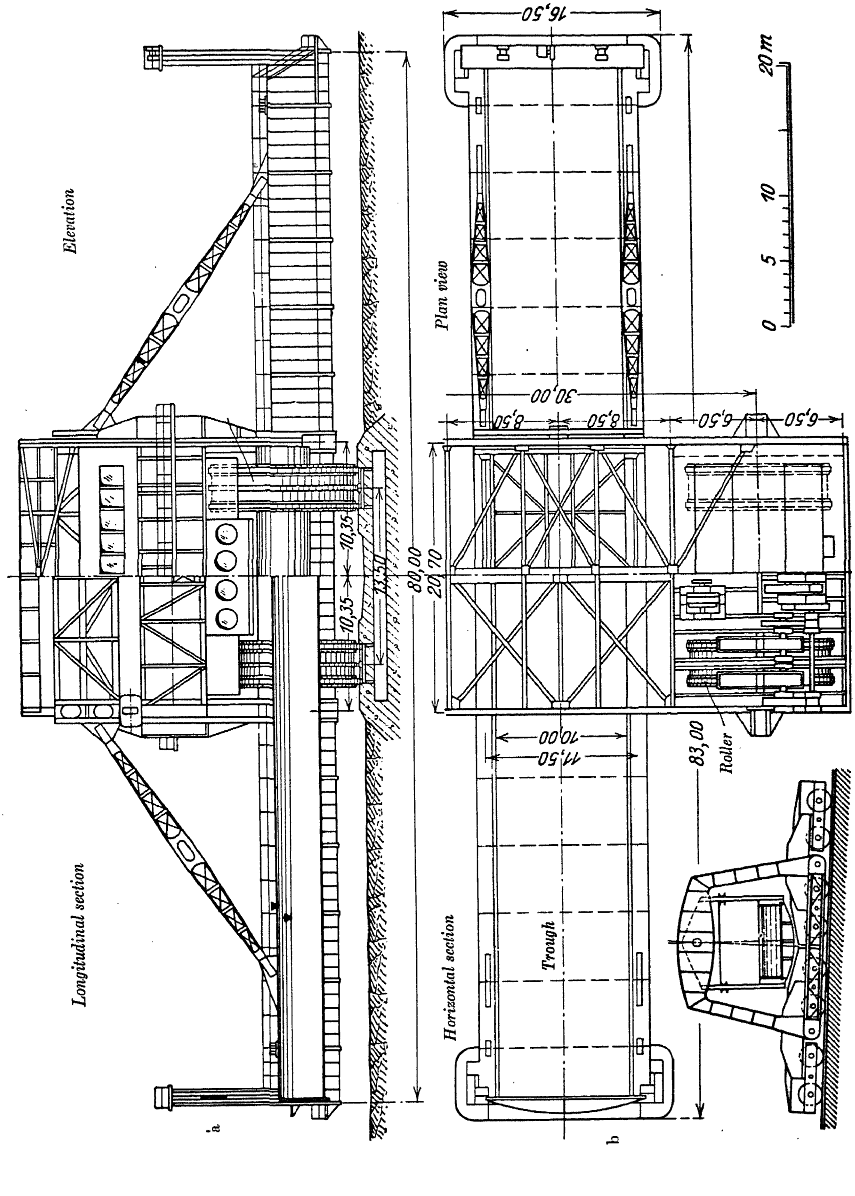


Fig. 479. Inclined section of the trough

In conclusion, the inclined plane of the M.A.N. will be described, the latter probably being the best design thus far proposed for inclined planes of this nature (Fig. 480). Here a trough is arranged on the center of a car

which was originally to roll on four giant wheels. Later, smaller wheels were provided. Instead of moving the car by cables having counterweights attached to them, it is to be operated directly by electric motors located on the car.



Figs. 480 a to c. M.A.N. plane

Cars are to be rolled on eight wheels provided at each side. The trough, according to the design of M.A.N., is to be supported in suspension in a double frame. It is also possible, as in the case of the *Oberlander Ebene* (Overland plane), to travel over an artificial water divide and to allow the trough to dip into the upper pool; however, it is preferable to provide bays. The arrangement proposed by M.A.N. seems to be particularly advantageous for especially large lifts. Fig. 481 indicates another form of trough design. Here the trough is hung in a large frame but is not intended to remain vertical in all positions, the water taking a different position with respect to the bottom of the trough after the latter moves over the sump. The frame rests upon four double wheel sets, which are connected in pairs to two balance beams so that the four left wheels and four right wheels, respectively, pendulate about two supporting points. The arrangement is simpler but the dead weight greater than the layout shown in Fig. 480.

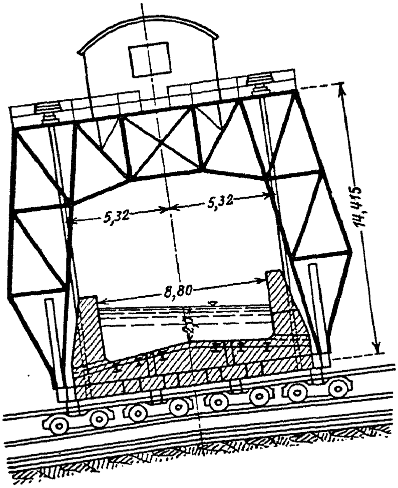


Fig. 481. Trough car on an inclined plane

b. Vertical Lifts

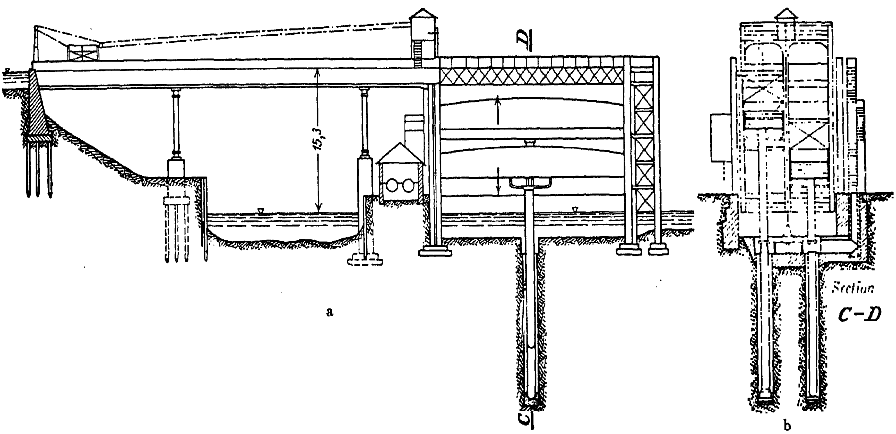
1. General

At the present time, all vertical lifts make use of a trough filled with water. Their speed of travel considerably exceeds that of the rate of rise possible for ships in locks. Thus, a mechanical lift has greater capacity for moving traffic than a shaft lock. The time required for entering and leaving is the same for both. There is a fundamental difference only when the trough is constructed for single ships and the shaft lock is constructed for entire tows. Technically, there is nothing to hinder the construction of mechanical lifts for completely coupled barge-trains. Whether or not a structure of this nature is economically capable of competing with large shaft locks must be investigated for individual cases. A great step forward would be made if it could be proven that wet conveying is an exaggerated safety measure. If it is possible to support ships in frames so that the pressure on each frame corresponds to the pressure exerted against the corresponding portion of the ship while

it is floating, more than half of the weight which the lift must raise could be eliminated. The layout would then be only about half as expensive and the lift would be a stronger competitor. In connection with lifts, no particular difficulty is encountered if the trough is constructed as indicated in Fig. 479. An arrangement of this nature may also be used for inclined planes. The manner of accounting for variations in water level is indicated in Fig. 483, which shows the layout of the Henrichenburg lift.

2. Hydraulic Lifts

Hydraulically operated lifts are one of the oldest types known. In these, the trough rests upon a hydraulically operated cylinder. Two troughs are arranged adjacent to each other, with their hydraulic cylinders coupled together. Fig. 482 shows the lift used at Anderton, England, before its reconstruction. (The new structure is shown in Fig. 490.)



Figs. 482 a and b. An early design of a hydraulic lift on the Weaver River at Anderton in England
a Elevation . b Cross-section through the trough

The water pressure is automatically attained by the weight of the trough. The movement is brought about simply by not allowing the water surface of the trough to come quite to the elevation of the canal water surface. The upper trough remains somewhat lower; the lower, somewhat higher. The upper trough thus receives a slight addition of water from the headwater canal, while the excess in the lower trough enters the tailwater. The connection between the hydraulic cylinders is closed before connection is made between the troughs and the pools. After the ships have left the troughs and others have entered, and the gates have again been closed, the upper trough, having a greater water content than the lower, forces the latter upward when the connection between the hy-

draulic cylinders is again opened. The speed of movement may be regulated arbitrarily by the valve between the hydraulic cylinders. This lift was comparatively small. The trough measured 1.5 by 4.7 by 22.7 m. (4.9 by 15.4 by 74.5 ft.) and was capable of conveying ships having a capacity up to 100 tons. The trough could be supported on one hydraulic cylinder because of the smallness of the layout. In the case of modern lifts which convey ships of 1,000 tons capacity, three or more cylinders would be required for each trough.

If the trough takes an inclined position, more water will have to be supported on its lower end, causing an excess pressure on the cylinder at this end in case several cylinders are used. In order to guard against accidents resulting in this manner, very accurate parallel guiding of the lift is necessary. Hydraulic lifts are now antiquated, although it is possible that with the modern improvements of machinery the hydraulic lift may sometime again provide competition to other types of lifts. A preliminary requirement is that the structure must be arranged as a single lift; that is, the hydraulic pressure must be generated by pumps if it is not desired to use counterweights. This thought leads to the use of lifts with counterweights, a type of structure which has been developed by various firms.

3. Float-Actuated Lifts

a. Lifts with Vertical Floats

Up to the present time, one float-actuated lift has been constructed, and a large number of float-actuated lift systems have been devised. The principal difference between the systems concerns the form of float; that is, whether it is in a vertical or horizontal position. The float lift which was built, and which still exists, has vertical floats. Situated at Henrichenburg, it provides ascent from the Dortmund-Ems Canal to the harbors of Dortmund. The lift has proved very satisfactory; however, no other structures of this design have been erected although in many cases similar circumstances exist. It has been shown that for such lift heights a mechanical lift is too expensive even though no supply water is available. Locks are considered a more economical type of structure, and the shaft lock has therefore been further developed.

Figs. 483–485 indicate the design of the Henrichenburg lift. The discussion concerning parallel guides for hydraulic lifts is also of importance in connection with float lifts. For example, if an excess of water is forced to one side of the trough by wind action, the load on the float concerned is increased and the float sinks deeper. Consequently, the trough lowers at this point and since the water keeps accumulating at one end as a result of the continued wind force, the difference in elevation of the ends

of the trough increases continuously unless some suitable sort of parallel guide is provided. The parallel guides are one of the principal features of the Henrichenburg lift. The trough rests upon five floats consisting of iron cylinders. These cylinders move in float shafts which are 9.2 m. (30 ft.) in diameter and 29.5 m. (96.8 ft.) deep. The substratum consists of firm marl. The upper portion of each of the shafts is lined with concrete, and the lower portion is lined with cast iron cylindrical segments which are coupled to one another. The bottom of the shaft is closed off

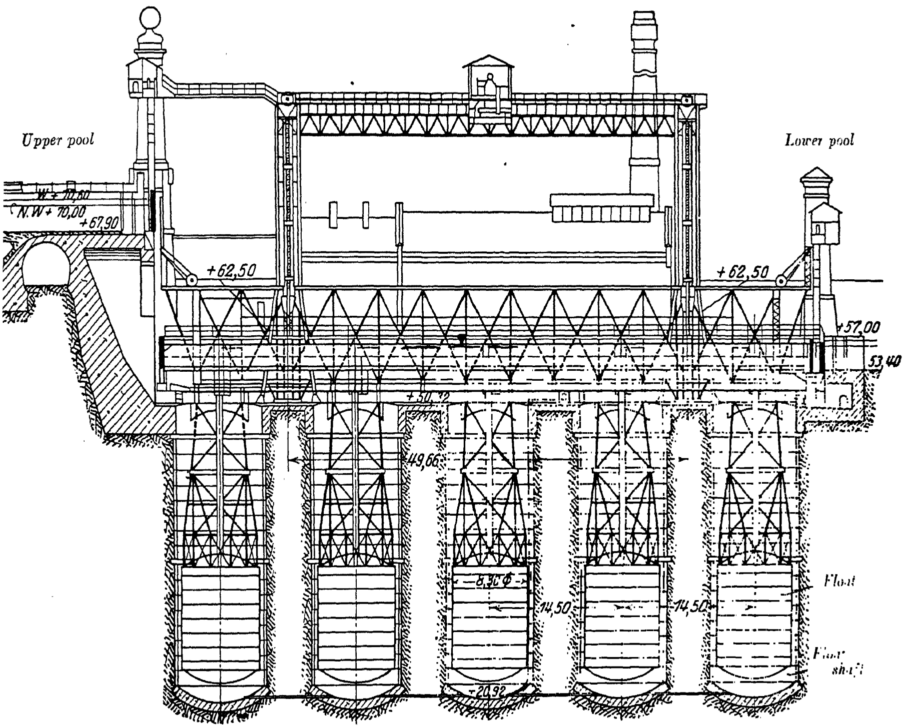


Fig. 483a. Henrichenburg ship lift. Longitudinal section along the axis of the trough

by spherical shaped caps 80 cm. (31.5 in.) thick. The shafts are connected to one another by horizontal pipes 12 cm. (4.7 in.) in diameter. The floats have an outside diameter of 8.8 m. (29 ft.) and a total height of 13.0 m. (43 ft.), thus being 1.5 times as high as broad. The total weight which the five floats are required to lift amounts to 3,100 tons, of which 1,540 tons consist of the water weight (or combined weight of water and ship when the trough is lifting vessels), 800 tons is the dead weight of the trough and its supports, and 760 tons the dead weight of the five floats. The trough moves between five trussed columns at each side, opposite

columns being connected to each other at the top as a portable. The screw sockets are fixed in position and do not turn; the spindles are turned from above so that they all revolve at the same speed.

Spindles have an outside diameter of 280 mm. (10.9 in.) over a length of 24.6 m. (80.7 ft.); the core diameter is 245 mm. (9.5 in.). They are held at 5 m. (16 ft.) intervals by movable toothed bearings which move upward with the trough. A core of 100 mm. (3.90 in.) diameter is bored

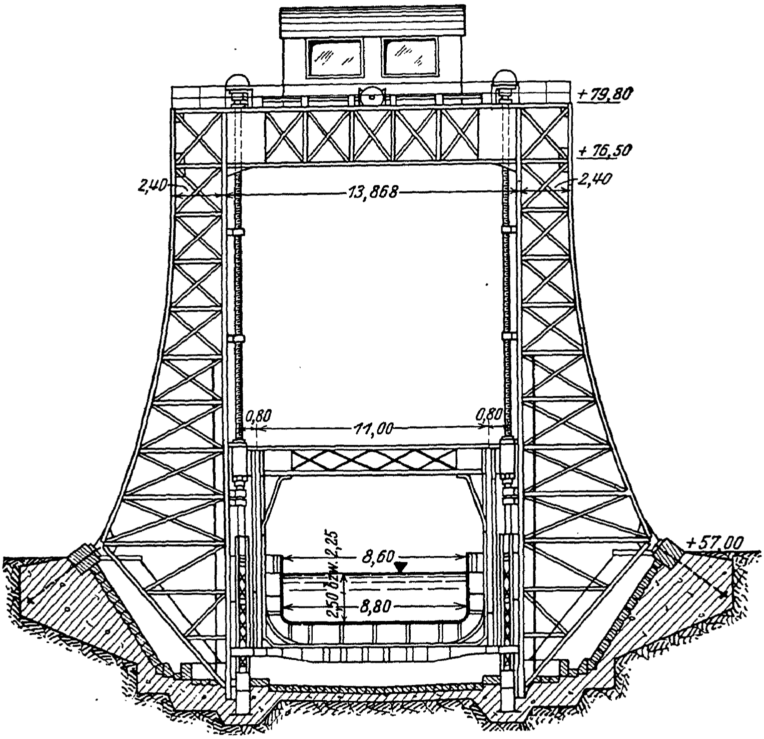


Fig. 483 b. Henrichenburg ship lift

Section through the chamber showing guides, scaffolding and operating house

from the center of the spindle. The female screws are 800 mm. (31.2 in.) high and made of bronze. They have proven satisfactory; however, as result of an investigation at the time of the World War it was found that the female screws were so badly worn that there was danger of failure to the structure. They were renewed in time to avert such an accident. Parts of this nature should be demounted at regular intervals in order that they might be adequately inspected.

The trough was constructed independently and hung from steel frame

girders, and is thus allowed to deform independently of the remainder of the structure. It is often kept cool because of frequent change of water, while the frames are subject to marked temperature changes due to the sun. The two parts, therefore, undergo very different temperature deformations. The trough has an inside length from gate to gate of 70 m. (230 ft.), an inside breadth of 8.8 m. (29 ft.); the useful breadth and length are reduced to 8.6 m. (28.2 ft.) and 68 m. (223 ft.), respectively, by

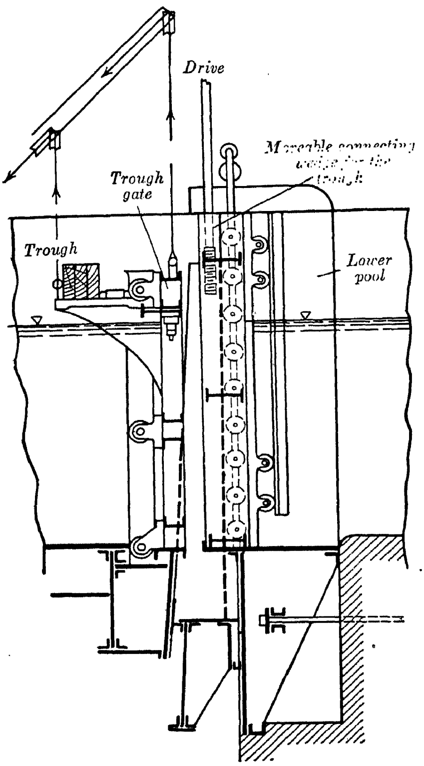


Fig. 484. Closing gates for the trough and pool of the lift at Henrichenburg

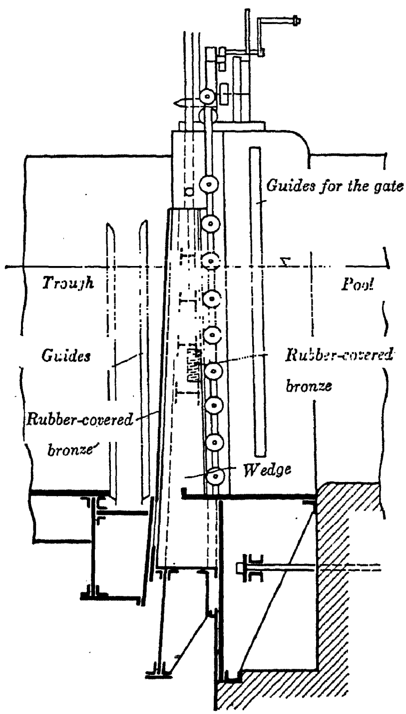


Fig. 485. Wedge closure at Henrichenburg

longitudinal washboards and transverse beams in front of the gates. On the outside the trough measures 71 by 10.5 m. (233 by 34.4 ft.). The side walls are 10 mm. (.39 in.) thick at the top and 12 mm. (.47 in.) at the bottom. Floor metal at the side is 14 mm. (.55 in.) thick, the curved and buckle plates 8 mm. (.31 in.) thick. The velocity of ascent and descent is 6 m. (20 ft.) per minute over an average distance of 14.5 m. (47.6 ft.) and a maximum of 16 m. (52 ft.).

The variation in headwater and tail-water level causes difficulty in

the case of all lifts. It is therefore necessary to make the wedge-shaped closure of the trough or pool movable vertically. This water-seal wedge operates on rollers as indicated in Figs. 484 and 485, so that it can follow the variation in water stage of the pool. If the water stage lowers, the wedge is lowered the same amount by means of a hand windlass. The rear surface of the wedge is connected to the pool by a rubber water-seal.

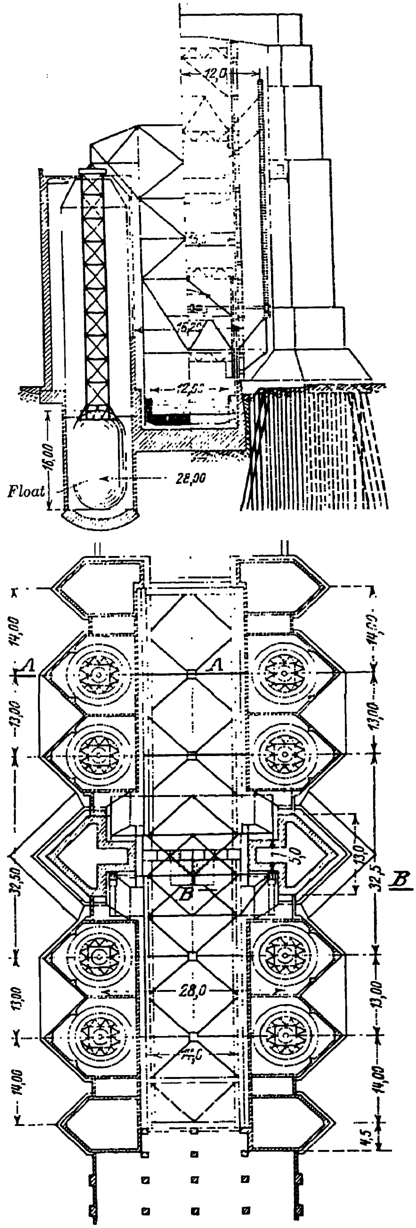
The lift gates of the trough and pool are coupled together when opposite each other. When the gate of the pool is raised, that of the trough is made to rise at the same time. Before the gates are raised, water is allowed to enter between them through a sluice valve. It has been observed that practically all rivet heads which have not been countersunk are worn off. As the ship leaves the trough, sand is drawn into the latter with the water at high velocities, and as the ship enters, it is forced outward again, causing considerable abrasion. Consequently only countersunk rivets should be used.

The ships have a length of 67 m. (220 ft.) with the rudder (62 m. (203 ft.) between outer edges of the stems), and a breadth of 8.2 m. (26.9 ft.) between washboards. For a loading of 600 tons they have a draft of 1.75 m. (5.74 ft.), under which condition there is a clearance of .75 m. (2.46 ft.) above the floor. According to Teubert, the same ships, with a length of 66.5 m. (218 ft.) overall and 8.2 m. (26.9 ft.) breadth across washboards, are capable of carrying 750 tons with a draft of 2 m. (7 ft.) and 900 tons with a draft of 2.3 m. (7.5 ft.). The dead weight of the ship is 180 tons. Unquestionably this lift might also be used for ships of 900 tons capacity.

An interesting feature of this mechanical lift is the relation of dead load to live load. The dead load is approximately 1,100 tons, the live load 2,000 tons, resulting in a ratio of approximately 1:2.

Inasmuch as the total load is counterbalanced, the power consumed in moving the lift is very small. For a corresponding lock without lift chambers a total dead load of $14.5 \cdot 8.6 \cdot 70 - 1{,}100 =$ approximately 8,600 tons would have to be moved; this load would have to be pumped as water to a height of 14.5 m. (47.6 ft.) in case there were an insufficient supply available for the upper pool. If the uplift by means of floats were not applied in the case of a lift, the dead load, including the ship, would amount to only about 2,300 tons. It is of importance that this load be counterbalanced by a float weight of approximately only 800 tons. The greater part of the performance consists in the construction of float wells. If a shaft lock is provided with thrift basins, approximately 74 per cent of the mass of water may be saved. There then remains a water weight of approximately 2,200 tons, including the ship, or about 1,100 tons without the ship; thus the quantity is substantially less than

for the lift described. It may, therefore, be concluded that a lift is theoretically superior to a shaft lock when the weight of the water which must be moved in the lock is greater than that which must be moved in the lift, since the water must be pumped. Practical determining factors which must be considered, however, concern construction difficulties, rapidity of operation, and the capacity of the mechanical lift. For a height of lift of 20 m. (66 ft.), the mechanical lift is probably superior to the lock. Certainty in this regard is possible only by accurate comparative computations. The Henrichenburg lift was constructed at a cost of 2.5 million gold marks.



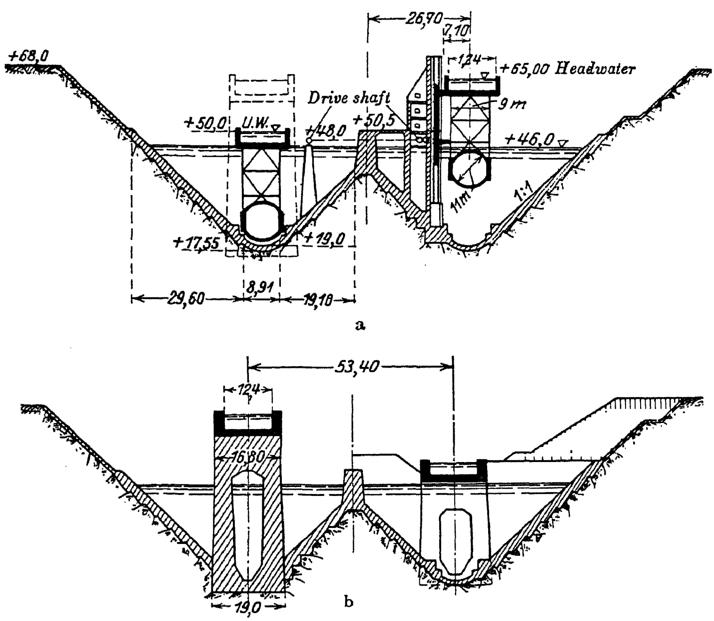
Figs. 486 a and b. Float actuated at lift at Harkort
a Vertical section and elevation b Plan view

The great depth required for the float pits led Harkort to propose arranging float towers at the sides of the lift. The float is then surmounted by heavy horizontal trusswork to which the trough is hung (Fig. 486). Instead of being placed underground in a well, the float is arranged in reinforced concrete towers extending above the ground. The cost incurred is much less than when underground wells are constructed. Furthermore, the trough has greater stability with a structure of this nature. No additional difficulty is incurred in providing parallel guides. The trough pit needs to be only a small amount deeper than the pit of the tailwater. Hence, no large foundation costs are incurred. The economy compared to a shaft lock must be ascertained for individual cases.

β. *Lifts with Horizontal Floats*

A mining engineer would consider the great depths required for float wells as an ordinary construction feature; the civil engineer would consider it a difficult foundation problem. Actually, no particular difficulties would be encountered in reasonably good substrata. In bad substrata extraordinary difficulties might be presented, leading to the design of horizontal floats under or at the side of the trough. A lift of this sort was first proposed by Offermann in a report made before the Ninth International Navigation Congress. Since then a number of firms have prepared various revisions of the original design.

One of the most recent designs of this type was prepared by the late Böhmler (Grün and Bilfinger) and by Harkort. Böhmler proposed providing three horizontal floats below the supporting structure of the



Figs. 487 a and b. Böhmler diving lock

a Section through diving chamber and guide tower b Section through head-bay and tail-bay

trough, the floats requiring but little vertical depth. Steel scaffolding is provided at the side and supports guide rollers (Figs. 487 a and b). The guide rollers are attached to a car which is intended to be operated mechanically.

Even for this type of lift (called "diving lock" by Böhmler) a depth

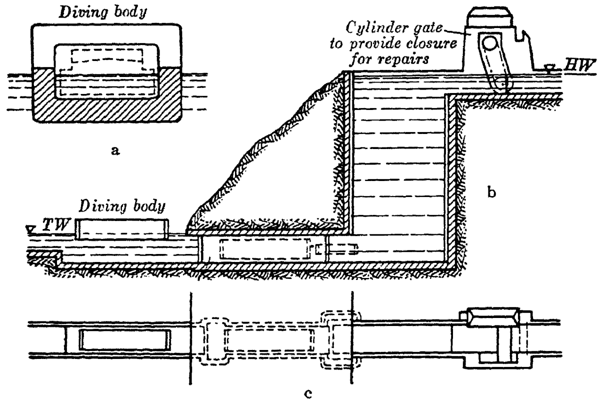
of $29.5-13+5=21.5$ m. would have been required for the conditions at Henrichenburg. To provide the 3,100 tons buoyancy necessary for the Henrichenburg layout, three floats of 70 m. (230 ft.) length and approximately 5 m. (16 ft.) outside diameter would be required. Compared to the Henrichenburg float height of 13 m. (43 ft.), therefore, a reduction in depth of only 8 m. (26 ft.) would be possible. As the steel structure with horizontal floats would doubtless require more space, the reduction in depth would probably not be more than 7 m. (23 ft.) and would require a pit much more difficult to construct. The pit of the lift proposed by Grün and Bilfinger provides for side slopes instead of fixed vertical walls. On the whole, it appears that horizontal floats do not possess any great advantage.

γ. *Diving Locks*

The oldest known attempt to construct a lift which would operate with the help of a float was made by the use of a diving lock. Robert Welden secured a patent in England, March, 1794, on such a lift. He constructed one for ships of approximately 22 m. (72 ft.) length, 2.13 m. (6.99 ft.) breadth and to surmount a difference in water elevation of 13.7 m. (44.9 ft.). The structure was destroyed but the time and manner of its destruction are not known. The diving lock consists of a shaft lock containing a horizontal float of such size that a ship may be placed within. The water level in the upper pool must be maintained at such a height that the float does not extend above the water when it is in a raised position. The float must be capable of being sealed against the side toward the pool to be served at both its upper and lower positions, in order to make passage between the float and pool possible. A similar diving lock was proposed by the Dutch Engineers Wouter-Cool and van Panhuis in a design contest of Prerau [lock lift of 36 m. (118 ft.) on the Elbe-Danube Canal]. The float is caused to sink by allowing water ballast to enter, and is guided by rollers fastened to the side walls. The dimensions are similar to the trough at Henrichenburg. The rise of the water stage in the pool loads the float, making it necessary to leave water out of the latter; similarly, ballast must be added when the water in the pool sinks. Since wind in the direction of the canal may generate a rise in the tail-water level and a drop in the headwater, difficulties are encountered which have not yet been eliminated. The float is intended to be sealed against the shaft walls by pressure from the inside of the float. The seal seems to be unsafe.

Böhmler proposed a new type of diving lock in order to overcome the difficulty of sealing the float; the float in the new type being provided with a giant shield which was to move upward and downward with the

float. This shield was to cover a vertical slit toward the tail-water side which was to extend from the bottom to practically the headwater level and keep the diving float in continual connection with the outside air. The construction of this shield, which is required at both ends of the float, seems possible; however, sealing an area of this magnitude for a travel distance of 10 m. (33 ft.) seems impossible.



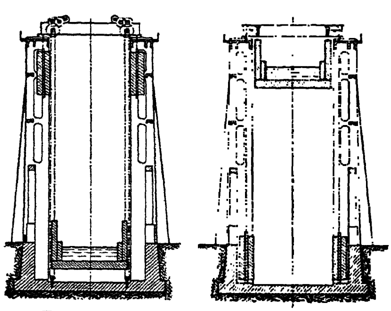
Figs. 488 a to c. Ship lift with diving shaft
a Diving chamber b Longitudinal section c Plan view

The success of submarines during the World War reacted favorably toward the introduction of this type of structure for inland locks. A design of this nature has been patented by M.A.N. The design is indicated in Figs. 488 a–c.

4. Mechanical Lifts with Counterpoises

a. Design by Demag, in conjunction with Gute-Hoffnungs-Hütter Dyckerhoff and Widmann, Siemens-Schuckert, and Baurat Roede, (Figs. 489 a and b).

This is a trough hung on cables from a large number of points, the cables being carried over sheaves at the top of the supporting structure and fastened to counterpoises at the opposite ends. In the design for Niederfinow, eight suspen sion points were provided, each of which is supported by ten cables, making a total of one hundred sixty supporting cables. Each cable is joined to an individual counterpoise. A frame is hung on each five of these cables, the frame extending below the counterpoises. In case one of the cables breaks, the counterpoise sits upon the frame so that the four remaining cables support all five counterpoises; in this way, the dis-



Figs. 489 a and b. Demag ship lift with counterpoises

tribution of counterweight acting upon the trough remains unchanged. The entire supporting structure is of simple reinforced concrete design. It is intended to operate the lift electrically with modern mechanical appliances. The large number of supporting cables makes inspection expensive. Wheels of 2 m. (7 ft.) diameter are required as sheaves for the cables in order that the latter will not be subject to too great flexural stresses. The structure may be considered one of the best, both for economy and satisfactory operation.

A lift having counterweights or cables has been constructed in England. The hydraulic lift at Anderton was reconstructed into one of this nature; each trough operates independent of the other (Fig. 490).

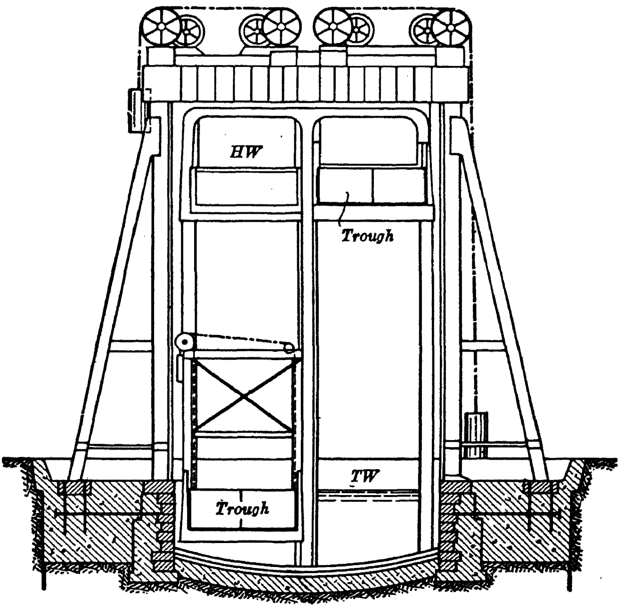


Fig. 490. Anderton ship lift in England after reconstruction

β. Lift by Bruno Schulz, Philipp Holzmann.

The objection of too many supporting cables, which are economical and structurally suitable but expensive to inspect, led to the construction of lifts with four counterpoises (Fig. 491). The design is so simple that the figure is self-explanatory. This type of structure is also used where settlement occurs due to mining operations, because settlement of the foundation does not affect the action of the counterweights; furthermore, the counterweight supports may be raised with jacks from time to time. These advantages are also obtained in the design of Demag.

γ. Lift by Bruno Schulz, Beuchelt (Fig. 492).

In this design two troughs 60 m. (197 ft.) on centers are hung from two huge double-armed levers. Sidewise swaying of the trough is hindered by two levers. Brake plates are provided in shafts, the shape of the shafts corresponding to the movement of the lever. The drive is provided by means of gears which operate in conjunction with a cylindrically shaped rack.

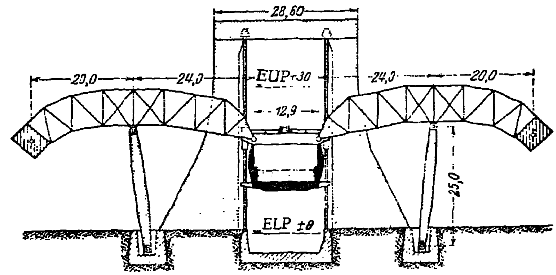


Fig. 491. Bruno Schulz ship lift with four counterpoises

Although very well conceived, the structure is of a somewhat complicated nature. In case one of the troughs is out of commission, neither

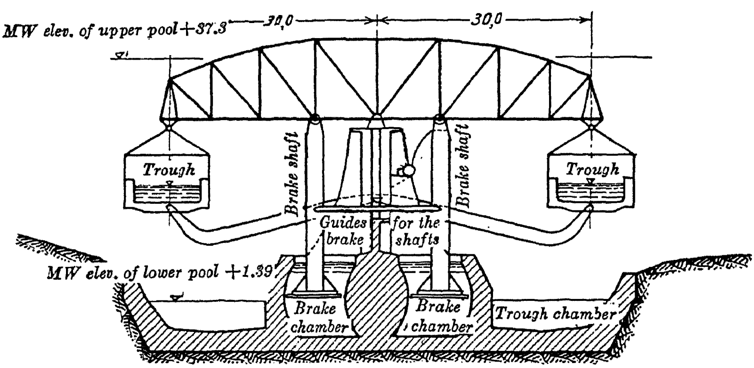


Fig. 492. Ship lift on cantilevers Bruno Schulz-Schnapp, Beuchelt

of the two can be operated. Such a design was planned for the 36 m. (118 ft.) lift of Niederfinow. However, because of the highly concentrated foundation pressures occasioned with this type of structure in the marl substrata encountered here, another type was finally adopted.

δ. Counterpoise Lift with Float by M.A.N., Gustavsburg (Figs. 493 and 494).

In this arrangement the trough is held by two large trussed levers which are supported on a float. The float may be arranged either in a container in the headwater (original proposal) or in a container in the

tail-water level (recent design). The trough is lifted through a guide scaffold as indicated in Fig. 493, the trough being lowered into the pool to the proper elevation. This procedure causes some diffi-

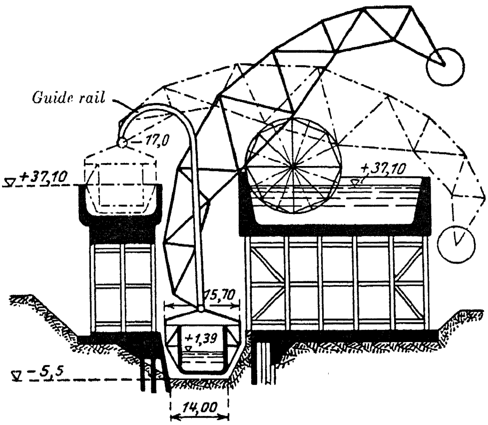


Fig. 493. Counterpoise ship lift (by M.A.N.) arranged with cantilevers supported on floats

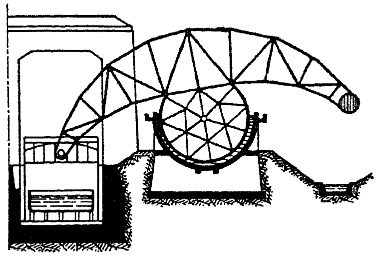


Fig. 494. M.A.N. ship lift with floating horizontal girders (vertical lift motion)

culty because of the large currents generated in the pool; it is proposed to remedy this condition by arranging large chambers in the trough, which may be used for receiving the water displaced. Large propeller pumps arranged for large capacities and low heads are intended for this purpose. The trough is provided with high side walls and end walls which are low enough so that the barge can travel over the ends after the trough has been submerged.

5. Drum Lifts

A line diagram of the drum type of lift is given in Fig. 495. A large drum of 52.6 m. (173 ft.) outside diameter floats in a basin of the tail-water and has two ring-shaped lock troughs. These troughs are kept closed during the time that the drum rotates. The design is very simple for a constant height of lift; however, it has been designed for a variable lift. In the case of a variable lift only the differences in the water stages come into consideration; the variations in the tail-water itself are not of importance because the drum raises and lowers to correspond to the stage of the water.

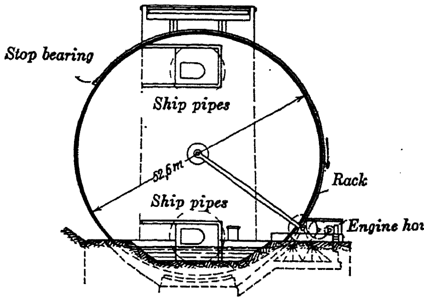


Fig. 495. Floating drum lock by Stokkert and Offermann of M.A.N. Designed in 1904 for a lift of 35.9 m. at Prerau on the Danube-Oder Canal

A. Comparison of Inclined Planes and Lifts, and a Comparison of These With Shaft Locks

Up to the present time shaft locks with thrift basins have been constructed for lifts as high as 16 m. (52 ft.) and, for such lifts, have proven themselves more economical than mechanical lifts and inclined planes. For large lifts, such as at Niederfinow, a mechanical lift is probably more economical than a flight of two or three shaft locks. It is not certain whether a Proetel lock or a by-pass lock flight would be more economical. A lift of about 40 m. (131 ft.) seems to be about the limit at which mechanical lifts can compete with inclined planes. Probably at this height of lift inclined planes become more economical.

The relative economy of the various systems has not been definitely ascertained. In order to make a complete analysis it would be necessary to investigate a large number of individual cases in which the foundation conditions, inclination of the territory, and size of ships vary. With such studies as a basis it would be possible to set up rules which, although not applicable to all cases, would provide a reasonable guide for most circumstances encountered.

K. FORE HARBORS; EQUIPPING AND OPERATION OF LOCKS

a. Location of Fore Harbors

Formerly fore harbors were arranged as indicated in Fig. 334 a, the outgoing ship following a rectilinear direction, the incoming ship then being required to maneuver from one side in order to enter the lock. More recently the outer harbor is arranged (Fig. 334 b) in such a manner that the incoming ships wait at some distance from the lock but lie in line with the lock axis so that they travel straight forward upon entering the lock; the outgoing ships must then take a curved path around the waiting ships. As the outgoing ship (or tow) is accelerating its velocity, it is readily steered and is capable of following a curved route more readily than a ship which must be retarded upon entering the lock. Fig. 335 a also shows the position of the locks in case twin locks are installed. Here the approach canal must be widened on both sides because of the narrow dividing wall; but, in case the middle wall is broad, the layout may be made the same as a single lock except that a double system would be used.

The fore harbors should be at least 1.5 times as long as a tow. The facility of operation of the locks depends largely upon these approaches.

b. The Equipping of Locks

The lockage time required for a single ship is made up somewhat as follows, there being little difference in the time required for lifts of 3 to 10 m. (10 to 33 ft.). For still higher lifts [such as 15 m. (49 ft.) at Minden] the time of filling must be increased slightly.

Entering and stopping of the ship	6.0 minutes
Closing lower gate	0.5 minutes
Fixing the ship and filling lock	5.0 minutes
Opening upper gate	0.5 minutes
Time required for ship to leave lock	5.0 minutes
Unforeseen conditions	3.0 minutes
Total time loss of a ship at a lock	20.0 minutes

This summary shows that the largest portion of the time is consumed in entering and leaving the lock, emphasizing the significance of towing arrangements in enhancing the quality of the lock. The following types are to be considered:

1. Capstans with cables.
2. Crabs upon elevated rails.
3. Towing locomotives.

Concerning number 1: Capstans are mounted on the ends of the locks; they pull the ships into the lock and are then used to retard the velocity of the ships so that the latter do not bump into the lock gates. As ships are ordinarily pulled from the bow, the ship may be readily held back by the capstan since the bow is usually over 80 or 100 m. (262 or 300 ft.) ahead of the capstan after having entered the lock.

Concerning number 2: The crab is moved along at such velocity that the lock attendant can readily walk beside it. He keeps the control rope in hand and can change the direction of travel within a moment's notice. This arrangement is also used for retarding the ship; however, it is recommended to install fenders in addition.

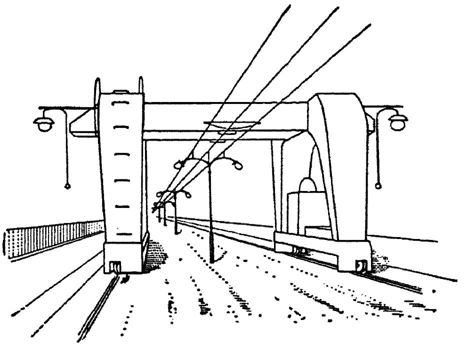
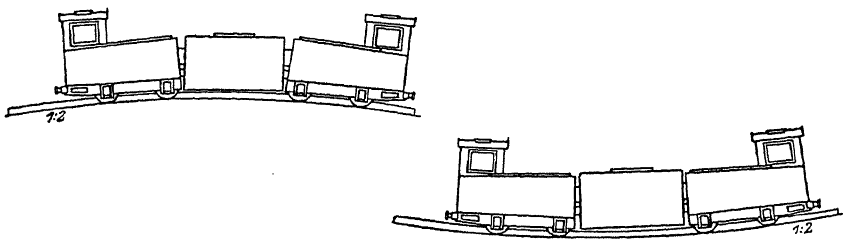


Fig. 496. German General Electric Company's design for a tow line locomotive at Hemelingen (Bremen)

Concerning number 3: Fig. 496 indicates the portal type of locomotive used for towing ships at the Hemelingen Lock. The operating motor is located near the wheels and the attendant rides on the locomotive. The

machine is so constructed that it can travel over the small service houses located on the center wall of the lock. The locomotive is also used for stopping the ship.

A more highly developed arrangement was adopted for the lock flights of the Panama Canal. Figs. 497 a and b indicate the locomotives used. In general, each ship is accompanied by four locomotives, one at each side of the front and one at each side of the back of the vessel; for large ships, as many as eight are used. They serve both for pulling the ship and stopping it. Each ship is pulled by the same series of locomotives through the entire flight of locks.



Figs. 497 a and b. Tow line locomotives at Panama

The locks should be provided with ladders so that the crew can come on shore. This is also necessary in order that the head boatman may communicate with the lock master. The lock master has complete authority concerning the manner of undertaking the lockage as he, rather than the captain, is responsible. Stairways are unnecessary in locks but should be provided in dry docks.

Fender timbers are not generally used in locks because it is necessary to have smooth walls without projections. A projecting edge on a lock wall might lead to tipping a ship if the latter becomes caught under this edge during the raising of the water level or remains hanging when the lock is emptied. In the case of fairly large lifts [over 3 m. (10 ft.)], crosses are required for attaching the hawsers to the lock wall. None of these may project beyond the edge of the wall. Crosses are preferred to rings because the latter would have to extend far into the wall in order to avoid projection beyond the plane of the wall. Crosses should not be staggered but should lie vertically above each other. In case they are staggered, the boatmen must move from one point to another in changing the hawsers as the ship rises. The vertical distance between crosses should be about 2 m. (7 ft.). An ordinary lock should have about four vertical rows of such crosses at both sides and spaced at about one-fifth the length of the chamber from each other. There should be a vertical row at least every 20 m. (66 ft.) along the length of the lock.

Ordinarily, in large sea locks intermediate mat fenders are used for absorbing impact. At the Panama Canal the use of impact buffers was adopted. Heavy spiral springs are constructed into the wall and faced with an impact timber to absorb the side impact of ships. The arrangement seems to be too sensitive to be recommended.

The rupture of the lower lock gate of Sault Ste. Marie due to being rammed, and similar damages to sea locks, led to the use of buffer chains which span the locks of the Panama Canal at both top and bottom. Double hydraulic cylinders are inserted in both walls of the fore locks which, when pulled apart, act as large pulley blocks. A heavy chain is wound over the sheaves of these sets of pulleys and usually hangs to the floor of the entrance so as not to disturb the passage of the ship. When the lower gate is under pressure, the pulley is tightened and winds up the chain so that it rises and closes the entrance. In case a ship travels into the chains a water pressure is developed in the hydraulic cylinders reaching 53 atmospheres (compared to 4.5 atmospheres when the chains are raised). A constant tension is generated in the chains amounting to 100 tons, the force being small at first and increasing rapidly until approximately 100 tons force is exerted in the longitudinal direction of the ship. Fig. 499 shows the pressure distribution for various sizes of ships traveling at various velocities. Thus, it will be observed that a ship of 10,000 tons displacement, if traveling at a velocity of 5 km. (3.1 mi.) an hour when reaching the chain, comes to rest in a distance of about 28 m. (92 ft.).

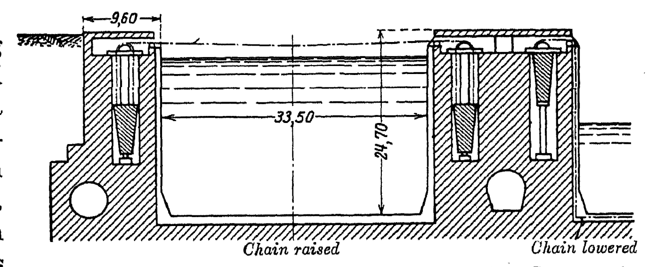


Fig. 498. Gatun lock; arrangement of the buffer chains. In a raised position at the left and a lowered position at the right

In order to bring the gates to a sealed closure rapidly, a gate closing machine was used on the Panama Canal, which grasps both mitering columns and presses them together. In the case of small locks or locks of intermediate size, machines of this nature are unnecessary, though they unquestionably serve the purpose of rapidly and safely closing the gates of the giant locks of the Panama Canal.

c. Lock Operation and Maintenance

1. Lock Operation

The personnel required at a lock is of great importance in connection with the economy of the layout. The old lock at Brunsbüttel of the

North-Baltic Sea Canal required one harbor master, two harbor overseers, two lock masters, eight lock attendants, one head mechanic, six mechanics, and six firemen. Operation at this lock took place day and night. In the case of a single lock more than half of this size crew would be required.

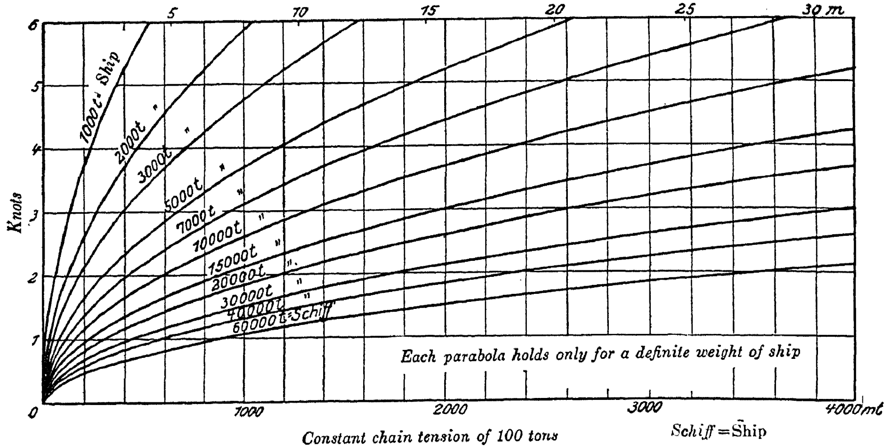


Fig. 499. Buffer chains; stressed condition of the chain

2. Maintenance

It is well to maintain well-pointed joints in all masonry work of the lock. The rapidity with which a sea lock deteriorates is astonishing if water is allowed to wash out the joints and enter the wall where it may accelerate the deterioration by the action of freezing and thawing. Of course, all exposed iron parts must be kept well painted. The life of a gate may reach 30 to 50 years if it is kept painted consistently. Coal tar paint has the advantage of sticking to the gate even though the latter is somewhat moist when the paint is applied. There are also many other satisfactory paints.

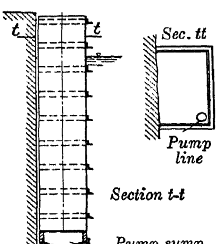


Fig. 500. Repair caissons

Underwater repairs can be accomplished with the help of a repair caisson constructed of reinforced concrete, the caisson being hung to the wall and the water pumped out. Such a caisson was used in Wilhelmshaven and other places for repairing rounded surfaces of the bays. Fig. 500 presents a schematic sketch of one of these caissons. The form of the water-seal is determined by a diver and then fastened to the caisson. The best type of water-seal consists of a rubber tube which is placed under water pressure after the caisson is lowered into place.

PART SIX — ARTIFICIAL WATERWAYS

A. GENERAL

a. Classification and Purpose of Artificial Waterways

Artificial waterways include canalized rivers,[1] inland canals, and sea channels. Strictly speaking, rivers regulated according to newer methods should also be considered artificial waterways, because in their entire make-up they differ so much from the natural condition that they can no longer be spoken of as natural waterways. For many of them only the discharge of water remains natural; the bed has become artificial. Nevertheless, it is customary to include the regulated rivers with the unregulated as natural water courses. Regulated rivers have therefore been treated in Part One. There is a characteristic difference between regulation and canalization of a river in that the aim of regulation is to bring about an equalization of the fall over long stretches frequently involving a concentration of fall at various short stretches (change from bend to transition), while in canalization the fall over large stretches is concentrated at various intervals.

At present both methods are frequently used consecutively for the development of a river system. The river is regulated as far as its discharge, especially at MLW, economically allows navigation by ships of customary size. Where regulation measures are impracticable because of the low rate of discharge, navigability is brought about by canalization. In the case of the Weser, for example (Fig. 540), by developing one intermediate pool between Bremen and Hanover-Muenden, regulation was introduced; a tributary, the Fulda, was canalized from this point to Kassel. The Elbe is regulated up to Schandau; it is canalized from Schandau upstream to Königsgratz, and the Moldau is canalized to Prague.

The Aller River is regulated from the Weser to the confluence of the Leine, and has been canalized from there to Celle. Unfortunately,[2] it is to be expected that the canalization of rivers will progress continually with the constant increase of transportation. Canalization of the Weser from Bremen to Hanover-Münden is already considered, and also the Werra from there still further upstream. Exceptions result from local

[1] According to Prussian Water Law, canalized rivers are legally classified as natural waterways. The distinction is clarified in that mention is made that a river, according to its origin, is a natural stream, while as a transportation way it may be called artificial.

[2] "Unfortunately" from the standpoint of the nature-lover.

conditions. Thus the Weser is first to be canalized from Minden to Bremen in order to tie Bremen with the Midland Canal and to replace the "genuine" Hansa Canal.

Canalization of the Rhine from Basel to Lake Constance is planned. The Main River, partly for the purpose of developing the Danube and partly in order to connect the large cities such as Würzburg, is to be canalized to Würzburg. A great attraction for extensive canalization is provided by the possibility of simultaneously developing water power. Today, navigation interests are still opposed to canalization of large rivers. It appears to be to the advantage of transportation interests to have canalization developed even on stretches of the Elbe where such measures are not yet contemplated. For example, if the Elbe were canalized from Magdeburg to Schandau, and the Oder from Stettin to Breslau, it might be possible to have transportation throughout the year over all German waterways from the Danube to the Rhine, Weser, Elbe, and Oder. Only the main stretch of the Rhine, because of its large rate of discharge and comparatively flat gradient, forms an exception (in Germany) to the necessity of canalization.

It is not economical to canalize rivers too extensively in their upper courses. Consideration of agriculture here necessitates too many low, short pools. Where the gradient of a river becomes too steep, for example where the slope exceeds[1] 1:1500, it is preferable to construct side canals or develop waterways transversely through the country without consideration of the river. It is thus planned to connect Leipzig to the Saale River with a special canal provided a connection directly with the Midland Canal is not preferable.[2]

A corresponding development which is beneficial to canals is effected by the installation of sea-ship canals. It is desirable to provide navigation for sea ships as far inland as possible, because the cost of transportation by sea is cheaper than any other means of transportation known. Thus there are a large number of sea-ship canals which connect large inland cities with the sea. Examples include Königsberg, Manchester, Brügge, Amsterdam, and other cities.

[1] The limiting slope of gradient must be determined for each river. The limit is principally dependent upon the depth to which the river has cut itself into the adjoining land. In very deep gorges canalization with pools of very great differences in elevation may be economical in conjunction with power development.

[2] This proposal was first made in 1916 by the author to the construction director, Rehder-Lübeck, and later led to planning the line by Rehder in his work *Der Nord-Süd Kanal und das zukünftige Mitteldeutsche Kanalnetz usw.* Lübeck, 1918. Printed by Borchers Bros., Lübeck.

b. Sea and Inland Ships

In all ships of today the transition from the former rudder ships to ships of rectangular cross-section has taken place. This has greatly influenced the design of lock entrances and chambers. This treatise does not discuss in detail the equipment of ships; this is left to books treating especially of ship construction.

The interrelation of length, breadth, and depth of various types of ships varies within relatively narrow limits. The variation of breadth to length of sea ships varies within the limits, $B:L = 1:6.7$ to $1:8.5$; that of depth to breadth, $T:B = .45:1$ to $.54:1$. A mean value for freight steamers is given by $L:B:T = 15:2:1$; for rapid transit steamers $L:B:T = 22.5:2.5:1$.

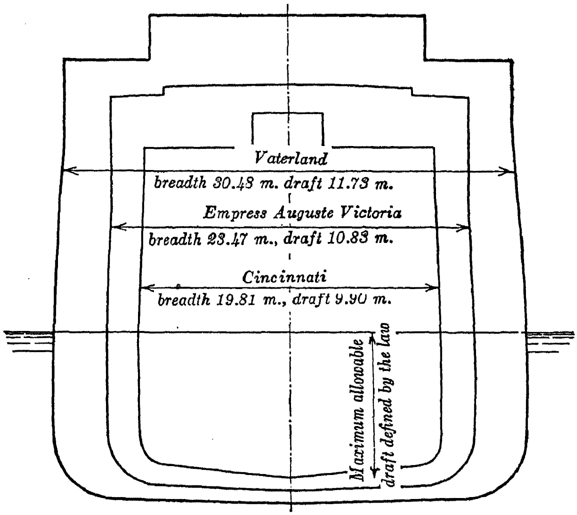


Fig. 501. Cross-sections of the sea ships of the Hamburg-American lines

The volumetric efficiency of the ship is the number by which the product $L \cdot B \cdot T$ must be multiplied in order to obtain the water displacement. This value δ varies in ocean freight steamers between .65 and .8; for rapid transit steamers, between approximately .58 and .63; and for large warships it is about .6. The displacement of a ship is thus given by the value $V = \delta \cdot L \cdot B \cdot T$ in which L is the overall length excluding the rudder, T the maximum depth extending below water, and B the maximum breadth in the water line excluding washboards.[1] Fig. 501 shows a few cross-sections of sea ships of the Hamburg-American line (before the World War). Fig. 502 gives a perspective with reference to the development of the lengths of ships of this organization from 1848 to 1915.

Inland ships differ from sea ships in that the former have a relatively small height compared to the breadth. Here also a rectangular cross-section is used. The ratios $t : b = 1 : 3$ to $1 : 6$ averaging $1:4.5$; $b : l = 1 : 5$ to $1 : 9$ averaging $1 : 8$, so that the average relation of length, breadth,

[1] The shipbuilder figures the ship breadth without and the hydraulic engineer with the washboards.

and depth is given by $l : b : t = 36 : 4.5 : 1$. The volumetric efficiency varies between .8 and .95. Canal ships, because of their low velocity, usually may be constructed with a higher volumetric efficiency than river ships. The latter, in consideration of currents and good steering arrangement, must be built comparatively slender.

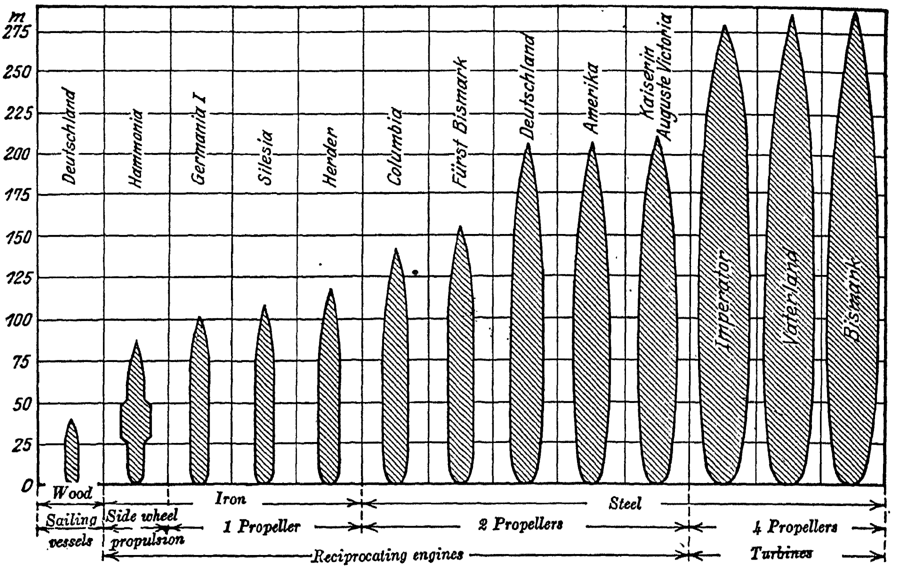


Fig. 502. Development of ship sizes of the Hamburg-American lines

The carrying capacity of inland ships varies between .75 and .82 of the water displacement; that is, the dead weight of the ship including equipment causes 25 to 18 per cent of the water displacement. The draft of empty barges varies between 25 and 40 cm. (between 10 and 16 in.). On rivers a much-used type of barge is the spoon-shaped barge in which the bow and stern are rounded in the form of a spoon. On canals, on the other hand, a sharp stem-form at the bow and stern has proved itself advantageous. Table No. 8 gives the usual dimensions of inland ships used in Germany.

The important developments of ships take place in periodic waves, largely dependent upon national economic conditions. After the application of iron and steel to the shipbuilding industry, ship sizes grew rapidly. Ships of over 4,000 tons capacity navigate the Rhine although there are only a few of this size on the river. At times the economic size is exceeded in the process of building ships to continually larger dimen-

TABLE NO. 8

USUAL DIMENSIONS OF INLAND SHIPS USED ON IMPORTANT EUROPEAN WATERWAYS

Waterway	Overall length		Maximum breadth		Draft				Weight Empty		Carrying Capacity		WaterDisplacement		Volumetric Efficiency
					Empty		Full								
	m.	ft.	m.	ft.	m.	ft.	m.	ft.	tons metric	tons English	tons metric	tons English	tons metric	tons English	
Weser..............	61	200	8.7	28.5	0.4	1.3	1.9	6.2	160	176	650	715	810	891	0.81
Elbe...............	75	246	10.6	34.8	0.39	1.28	2.0	6.6	250	275	1090	1199	1340	1474	0.87
Rhine (large ship)....	87	285	11.1	36.4	0.47	1.54	2.6	8.5	360	396	1760	1936	2120	2332	0.87
Oder (Breslau).......	55	180	8.0	26.0	0.40	1.31	2.0	6.6	150	165	610	671	760	836	0.89
Dortmund-Ems Canal.	67	220	8.2	26.9	0.45	1.48	2.42	7.94	180	198	940	1034	1120	1232	0.88

sions; there then results a definite relapse which is again followed by a lively growth, etc. On the German rivers and canals the number of very large ships is comparatively small. However, the 2,000-ton ships on the Rhine will probably soon gain greatly in significance; on the Elbe the 1,200-ton ships will probably become predominant, while the canals are being arranged to accommodate 1,000- to 1,200-ton ships. Official limitation of ship sizes is now being introduced in Germany. It has been decided by statute of the shore states of the Elbe to limit the breadth of Elbe barges to 11 m. (36 ft.) and their length to 76 m. (249 ft.). Engels recommends that ships of 1,000-ton carrying capacity be given a breadth of 9.2 m. (30 ft.) inclusive of washboards, a length of 80 m. (262 ft.), and a draft of 1.9[1] m. (6.2 ft.) at full loading.

Attempts have also been made to construct ships of reinforced concrete so as to obtain considerable reduction of resistance to propulsion. A smoothed cement surface provides a lower frictional factor to water than a metal surface, but reinforced concrete ships have thus far proven to be inferior in strength.

c. Ship Resistance

1. Influence of the Form of Cross-Section on the Resistance

If the breadth of a waterway is at least 15 times the breadth of the ship and the depth at least 20 times the draft, according to experiments by Gebers, the resistance is practically the same as though the water boundaries were unlimited. Accordingly, navigation in the ocean may be considered as navigation in a water course of unlimited cross-section.

The development of sea navigation in the past 50 years is based principally on experiments on ship resistance. The first important experiments of this nature were made by Froude. These indicated that the resistance to movement varied approximately as the square of the velocity. The resistance did not increase directly as the ship length, but rather at a smaller ratio. Investigations also indicated that the exponent of the velocity was dependent upon the length of the ship.

For ships traversing rivers and various types of canals, unlimited boundaries of the water course can not be assumed because, in the case of canals, the breadth is actually only about three to five times that of the ship and the useful depth only a fraction greater than the draft of the ship.

Inland navigation divides itself into two classes: the navigation of sea ships in sea canals, and of inland ships on canals and rivers. Since sea ships are used principally in navigation of the open sea, and their operation on canals is exceptional, the cross-section of sea-ship canals

[1] *Z. Binnensch.* 1922, p. 281.

must be designed to suit the form developed for the ship. The form of canal section must satisfy the following conditions. It must:

1. Guarantee safe navigation,
2. Involve low construction cost,
3. Provide low resistance to ship movement, and
4. Allow for future extension.

The third requirement is secondary in connection with sea canals.

The inland ship navigates only canals and improved rivers; consequently, the shape of the barge and the form of bed of the waterway should be determined to suit each other. The barge form and the form of cross-section of inland waterways should also be selected to fulfill the above enumerated requirements, but in this case the ship and waterway are made interdependent. Literature has hitherto dealt principally with resistance to navigation; the other three requirements have hardly been considered. However, all four points of view should be given consideration and ship resistance alone should not govern. Solely emphasizing the importance of ship resistance has led to the construction of canals which can be improved only by overcoming some of the one-sided viewpoints which have been fostered heretofore.

Because of the two-fold character of the problem, the correct development of inland canals is more difficult than that of sea canals. Furthermore, the design of sections for inland canals is more intricate than for river canalization, because the cost of earthwork involved in canal construction usually necessitates substantially smaller cross-sections than are available in rivers.

Ship forms which come into question at present are the sharp stem and the spoon form. Apparently, according to experiments, the sharp form deserves preference on canals. On the other hand, no clarification has been offered concerning the best form of section for canals. The trough form in which the cross-section is similar to a parabola has long been thought the most favorable form. This is the form used as standard for the Midland Canal (Germany) between Bevergern and Hanover. According to experiments by Froude and later by Engels, the ship resistance in a limited canal section increases according to the relation $v^{2.25}$; also the relation $n=F/f$; that is, the ratio of the canal cross-section to the submerged cross-section of the ship must become larger as the ship velocity increases. At a velocity of 1.5 m. (4.9 ft.) per sec. [5.4 km. (3.4 mi.) per hr.], a value for $n=5$ to 6 should exist; but when the velocity is augmented to 2 m. (6.56 ft.) per sec. [7.2 km. (4.7 mi.) per hr.], n must be increased to 10 if the resistance to the ship is not to become excessive. Of two equally large wetted cross-sections, the wider generates the greater ship resistance.

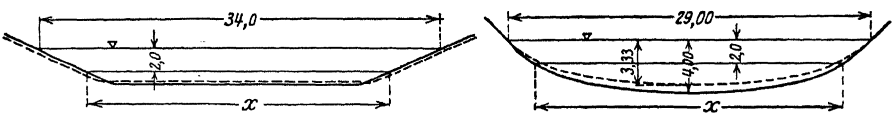
It seems apparent from experiments made in Dresden by Engels, that the trapezoidal form of section provides less resistance at ordinary navigation velocities than the trough form.[1] In experiments made by Thiele[2] in the Berlin laboratory, this point of view regarding equal cross-sectional areas was again verified. Experiments with sharp stems indicated that with a draft of 1.5 m. (4.9 ft.) the trapezoidal cross-section is more favorable than the trough-shaped section up to a velocity of 1.9 m. (6.2 ft.) per sec. [6.8 km. (4.2 mi.) per hr.]. For a draft of 1.79 m. (5.87 ft.) the trough becomes better at a velocity of 1.2 m. (3.9 ft.) per sec. [4.3 km. (2.7 mi.) per hr.] and the advantage increases as the velocity becomes greater. With a draft of 2.07 m. (6.79 ft.) the trough cross-section was superior in all cases. Experiments in which the spoon form of boat was used showed the trough to be superior under all circumstances; the superiority with a draft of 1.5 m. (4.9 ft.) and velocities of 1.5 to 2.07 m. (4.9 to 6.79 ft.) per sec. was only 6 to 10 per cent. From these experiments it was concluded that the trough is the most favorable cross-section for canals, but this conclusion is not valid, as ships with sharp stems (because of larger capacity) are more economical than ships of the spoon form. Experiments with the spoon form, therefore, are not conclusive. Further studies should be systematized according to stretches in flat country with embankments and stretches in deep cuts, since construction costs are strongly influenced thereby. In the first case, the size of the wetted cross-section may be of decided importance; the breadth also influences the resistance materially, particularly the breadth at the elevation of the submerged ship-bottom. This breadth is of decided importance to navigation because the frequency of scraping the side slope is largely dependent thereon. Safety of the canal shore, especially in the embankment stretches, is more important than the attainment of a somewhat smaller resistance to transportation. Therefore, the useful breadth at embankment stretches should be considerably enlarged with mild transitions, especially since the additional cost involved therein is usually small. It is not proper to compare sections such as those in Figs. 503 and 504, because if it is possible at the water line, where the strongest attack occurs, to provide a slope as steep as that of the trough, the same is also possible for the trapezoidal profile. In flat country, cross-sections should be compared which possess the same breadth at the elevation of the canal bottom; in this case, equal cross-sectional areas are not taken as the wetted cross-section but as the total excavated section, because *canal costs do not depend upon the length of*

[1] Translator's note: There is considerable controversy in Germany concerning the "correct form of cross-section" for canals. The views presented here are those of the author.

[2] Thiele, *Schiffswiderstand auf Kanälen,* 1904. *Z. Binnensch.*

the wetted perimeter but upon the amount of excavation. The trapezoidal section then is deeper than the trough section, it has a narrower surface water breadth than the trough section, and consequently it provides less resistance to transportation. It is to be pointed out that the difference in resistance is very small in nature, so that this advantage falls out of consideration. As for the water movement as a result of deeper sides, the trapezoidal section is probably the better because a smaller wave develops at the shore of this form when ships navigate the canal.

The relation of the two sections is indicated in Figs. 505 and 506. The advantage of the trapezoidal section is still more evident in deep cuts, because here the amount of excavation is practically dependent upon the water surface breadth. A further influence favoring the trapezoidal section is the cost of construction of the many bridges and inverted siphons, which become smaller the narrower the canal.



Figs. 503 and 504. Cross-section of the Uebigau experimental canal

A proposal by Krey may be of interest in connection with the foregoing discussion. Krey investigated the cross-section represented by Fig. 506. To avoid the large surface breadth he proposed fixing the shore by bulwarks as indicated in the figure. This restriction of the section results in a reduction of ship resistance of about 5 to 6 per cent with a velocity of 5 km. (3.1 mi.) per hr. and about 9 per cent at 6 km. (3.7 mi.) per hr. About 8,000 cu. m. (10,400 cu. yd.) of reinforced concrete wall with anchoring would have to be constructed for 1 km. (.621 mi.) of canal, while with the surrounding country 3 m. (10 ft.) above the water level 27,000 cu. m. (35,200 cu. yd.) of earthwork would be saved together

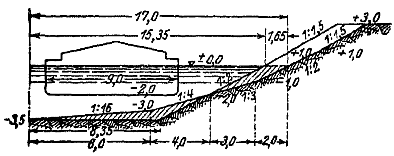


Fig. 505. Comparison of trough and trapezoidal sections

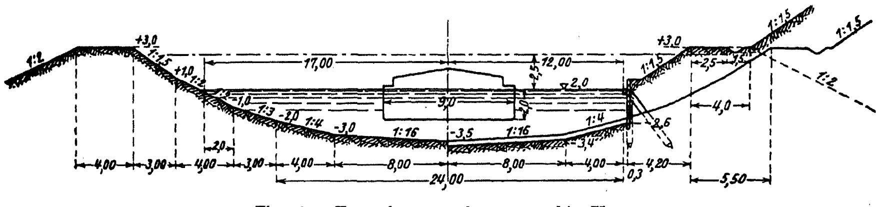


Fig. 506. Type of cross-section proposed by Krey

with 10,000 sq. m. (108,000 sq. ft.) waterproofing area; the ground area required would be reduced by about 11,000 sq. m. (119,000 sq. ft.) and the protection works by 9,000 sq. m. (97,000 sq. ft.). There is also a shortening of 13 m. (43 ft.) for bridges and 11 m. (36 ft.) for inverted siphons. Krey considered the cross-section with vertical walls economically sound for competition with other sections. This section seems to be the correct form for deep cuts.

According to an article by Schmidt,[1] the advantage of Krey's proposal on the Dortmund-Ems Canal can be realized only with a cut of at least 34 m. (112 ft.) depth to the water surface, making the proposed layout entirely out of the question for normal construction.

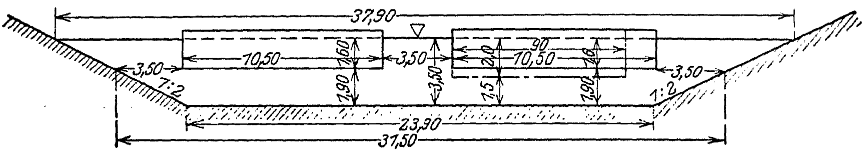


Fig. 507. Trapezoidal canal cross-section

The trough shape is entirely unsatisfactory for water-tight embankment stretches because subsequent widening of the canal is impossible, while a cross-section with a flat bottom and corresponding arrangement of the waterproofing allows such widening.

In conclusion it is again emphasized[2] that considerations of operation and, above all, low cost of construction, are as important as resistance to ship motion. Consideration of the use of trapezoidal sections in connection with construction procedure led to further development of the Midland Canal, including the stretches Hanover-Hildesheim and Hanover-Pein which were formerly projected as trough-shape sections.[3] Recent projects have indicated that a regular cross-section such as Fig. 584 is to be preferred to the trough section.

Experience on the Dortmund-Ems Canal, indicating that the angle of slope 1:2 is too steep, has probably only limited importance because the cross-section there is too small. For large cross-sections (Fig. 507), the velocity of the current and the resulting danger of caving is less, although depending upon the nature of the ground traversed.

2. Magnitude of Ship Resistance

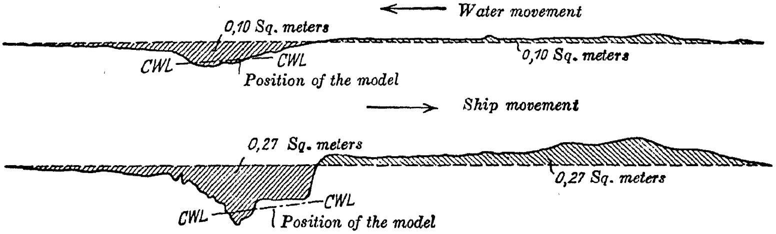
The magnitude of ship resistance was clarified particularly by experiments made by Froude (1871), Sweet (Erie Canal, 1877–78), de Maas

[1] Oberbaurat Schmidt, *D.W.W.*, 1927.

[2] Franzius, Die Querschnittsausbildung von Kanälen. *Z. Binnensch.*, 1926; Discussion *Z. Binnensch.*, 1927, Vol. 4.

[3] Paul Schmies, *Doctorate Dissertation*, Technical University of Hanover, 1925.

(Seine, etc. 1890–95), Haak (Dortmund-Ems Canal, 1900), Mattern and Buchholz (*Grosse Schiffahrts-Weg Berlin-Stettin,* 1912), Gebers and Engels.



Figs. 508 and 509. Profile of the water surface taken during the movement of a barge model having a length of 7 m. and moved at a velocity of .474 m. per sec. and .674 m. per sec. respectively (the scale of lengths is 1/10 that of the scale of heights).

The position of the water level may be determined by special arrangements, for various velocities of a ship. Figs. 508 and 509 were obtained by making such measurements for velocities of a ship model of .474 m. (1.55 ft.) per sec. and .674 m. (2.21 ft.) per sec., respectively, corresponding to velocities in nature of 5.12 km. (3.18 mi.) per hr. and 7.28 km. (4.52 mi.) per hr. The distorted diagrams show a rise of the water surface to occur in front of the ship, corresponding to a lowering of the level in back of the ship. In moving forward, the ship must continually displace a quantity of water depending upon its cross-section and velocity. A valley is created in the water behind the ship, while in front of it a water mound is formed. Water heaped up in front of the ship must continually flow past it into the valley behind it. The amount of rise of the water in front of the ship must be great enough to create a pressure head sufficient to generate the necessary water velocity. The mound crest in the front portion of the backwater surface was due to the irregularity in the testing flume, causing a severed wave to precede the ship. The experiments show that the ship resistance is not dependent merely upon its velocity, but upon the sum of the ship velocity and return-current velocity.[1] According to Gebers, in navigation canals with quiet water and as long as the depth below the bottom of the ship is greater than 1 m. (3.28 ft.), the towing resistance is approximately

$$W=(k\cdot f+\xi\cdot 0)v_r^{2.25} \text{ in } kg.^2$$

[1] When moving, a ship thus sinks lower than its static elevation in the water; for a series of towed barges each succeeding ship sinks somewhat lower than the foregoing. On the Dortmund Canal lowering amounting to as much as 11 cm. (4.3 in.) has been observed; in sea canals the sinking may amount to as much as 1.5 m. (4.9 ft.); this amount is reached in the Suez Canal. Strombaudirektor Plate indicates that on the lower Weser between Bremen and Vegesack, ships of about 7 m. (23 ft.) draft sink .5 to .7 m. (1.6 to 2.3 ft.). (Weserztg. of Feb. 15, 1925, No. 84, first supplement.)

[2] Engels, *Handbuch des Wasserbaues,* Third Edition, Vol. 2, p. 1005.

in which $v_r = \dfrac{v \cdot (f+f_s)}{F-(f+f_s)} + v$, and $f_s = B \cdot s$ (cross-sectional area of the drawdown);

$$s = \frac{(v+v_1)^2 - v^2}{2g} \text{ (drawdown)}, \; n = \frac{F}{f}; \; v_1 = \frac{vf}{F-f} = \frac{v}{n-1}.$$

In these relations B is the cross-sectional area of the barge, F the cross-sectional area of the canal, v the navigation velocity of the ship, k the bow coefficient of the barge, ξ the coefficient of skin friction, the latter being .14 for new steel ships and about .3 for old wooden ships.[1]

If the water depth below the ship is less than 1 m. (3.28 ft.), the formula becomes more complicated. The same is true when navigation on water courses having flowing water is considered.

Newer extensive experiments on full-scale equipment have been undertaken by Mattern and Buchholz on the waterway between Berlin and Stettin.[2]

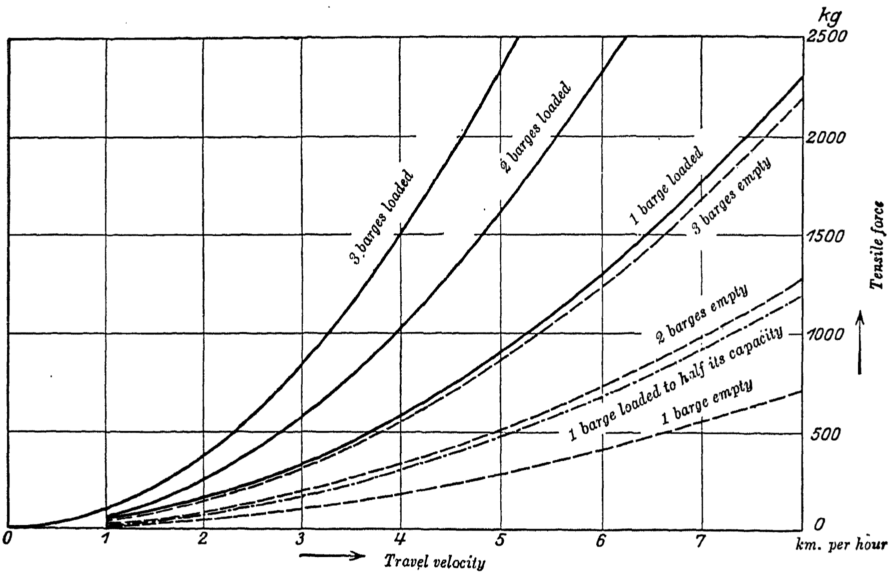


Fig. 510. Tension in the hawser for various travel velocities

The experiments by Mattern and Buchholz contain the most important results of recent times regarding barge trains. Experiments were

[1] De Thierry rightly points out that wooden ships, like blunt ones, should have to pay a higher towing price, and blunt wooden ships the highest rate.

[2] Mattern and Buchholz, *Schlepp- und Schraubenversuche im Oder-Spree-Kanal und Grosschiffahrtsweg*, 1912, Engelmann, Berlin-Stettin.

made on a stretch 1 km. (.621 mi.) long, the cross-section of which is shown in Fig. 523. The experiments were made at a somewhat raised water stage with about 72 sq. m. (775 sq. ft.) wetted cross-sectional area. The cross-sectional area of the submerged part of the ship was about 14 sq. m. (150 sq. ft.), giving a ratio between ship and canal sections of 1:5.15. A long series of experiments were made, the results of which are presented in Figs. 510 and 511 and in Table No. 26.

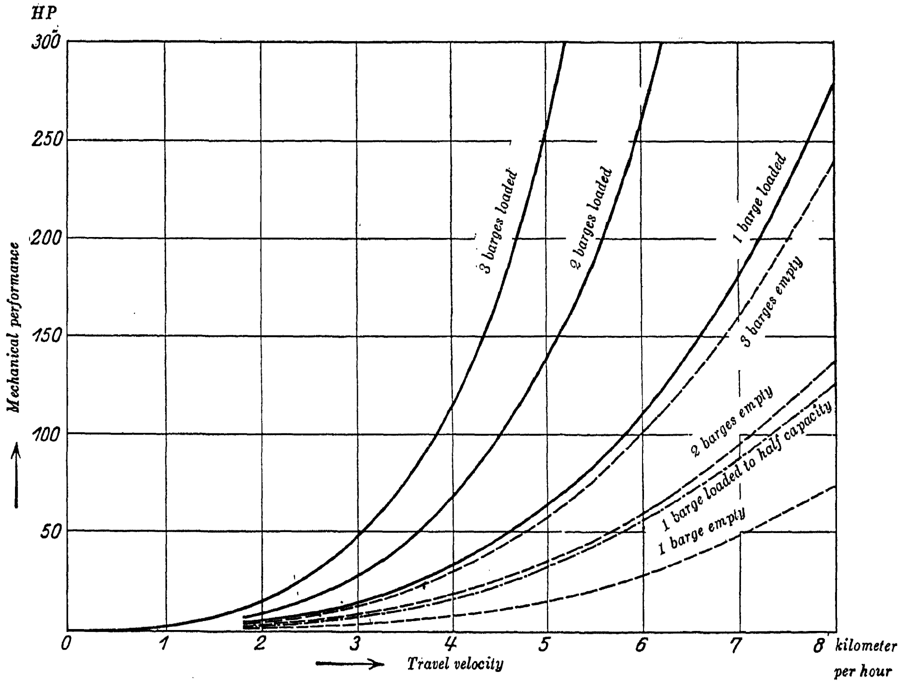


Fig. 511. Mechanical performance for various travel velocities. Dimensions of barges: length 65 m., breadth 8 m., draft when loaded 1.75 m.

The figures and Table No. 9 give a summary of the towing forces and corresponding mechanical performance; the efficiency of the towing performance is evident. The mechanical performance does not increase directly as the velocity in all experiments. For example, with a barge traveling at 2 km. (1.2 mi.) per hr., 4.5 HP are necessary, at 4 km. (2.5 mi.) per hr. 33 HP, and at 8 km. (5 mi.) per hr. 285 HP. Thus, with an increase of velocity from 2 km. to 4 km. (from 1.2 to 2.5 mi.) per hr., the mechanical performance rises from 4.5 to 33 HP, that is, 7.4-fold, and for a velocity increase from 4 km. to 8 km. (from 2.5 mi. to 5 mi.)

TABLE NO. 9

Results of Experiments on Ship Resistance

(Metric Units)

Gross Load	Velocity in km. per hr.	Velocity in m. per sec.	kg.-m. per sec. $\frac{Z \cdot v}{75}$	HP	$\frac{HP}{\frac{Z \cdot v}{75}}$	Efficiency η $\frac{\frac{Z \cdot v}{75}}{HP}$
1 Barge empty 186 tons	2	0.555	0.306	1.0	3.27	0.306
	3	0.833	1.00	3.0	3.00	0.333
	4	1.111	2.59	8.0	3.09	0.325
	5	1.388	5.10	16.0	3.14	0.318
	6	1.666	8.90	29.0	3.26	0.307
	7	1.944	14.22	48.0	3.37	0.294
	8	2.222	21.00	76.0	3.62	0.276
1 Barge half loaded 392 tons loading 206 tons	2	0.555	0.630	2.5	4.02	0.250
	3	0.833	1.772	7.0	3.95	0.251
	4	1.111	4.29	17.0	3.96	0.252
	5	1.388	8.53	32.5	3.81	0.262
	6	1.666	14.65	57.0	3.89	0.257
	7	1.944	23.80	88.0	3.70	0.270
	8	2.222	35.60	127.0	3.57	0.280
1 Barge loaded 786 tons loading 600 tons	2	0.555	1.12	4.5	4.02	0.249
	3	0.833	3.62	14.0	3.87	0.258
	4	1.111	8.50	33.0	3.89	0.257
	5	1.388	16.70	65.0	3.90	0.257
	6	1.666	28.60	113.0	3.95	0.253
	7	1.944	45.50	184.0	4.05	0.247
	8	2.222	67.80	285.0	4.20	0.238
2 Barges empty 372 tons	2	0.555	0.640	2.5	3.91	0.256
	3	0.833	2.060	7.5	3.64	0.275
	4	1.111	4.81	18.0	3.74	0.267
	5	1.388	9.23	36.0	3.91	0.256
	6	1.666	15.96	61.0	3.82	0.262
2 Barges loaded 1,572 tons loading 1,200 tons	2	0.555	1.93	8.5	4.40	0.227
	3	0.833	6.33	28.0	4.42	0.226
	4	1.111	14.95	68.0	4.54	0.220
	5	1.388	29.80	142.0	4.76	0.210
	6	1.666	49.00	263.0	5.37	0.186
3 Barges empty 588 tons	2	0.555	1.039	4.0	3.85	0.260
	3	0.833	3.33	12.0	3.61	0.278
	4	1.111	8.14	30.0	3.68	0.272
	5	1.388	15.85	59.0	3.72	0.269
3 Barges loaded 2,358 tons loading 1,800 tons	2	0.555	2.75	15.0	5.45	0.183
	3	0.833	9.43	48.0	5.09	0.196
	4	1.111	22.40	116.0	5.18	0.193
	5	1.388	43.20	254.0	5.88	0.170

per hr., the rise is from 33 to 285 HP, or 8.6-fold; hence, the power required increases much more rapidly than the square of the velocity. The numerical table is of particular value because it shows the variation of performance for a large variety of circumstances. It is clear that an increase in velocity to over 5 or 6 km. (3.1 or 3.7 mi.) per hour necessitates application of great mechanical performance. The same

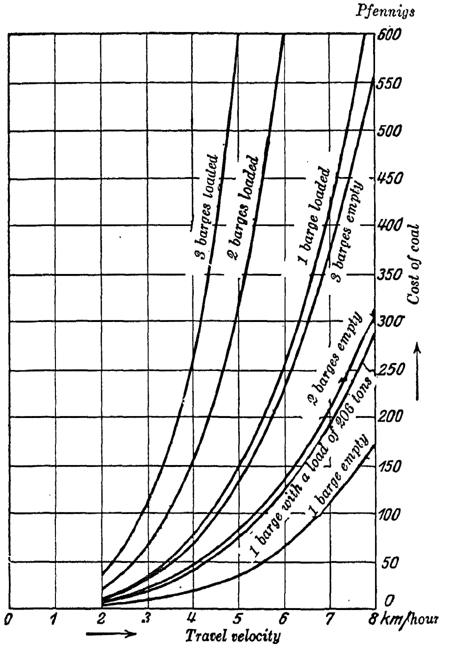


Fig. 512. Cost of coal for 1 hour of towing for various velocities of travel

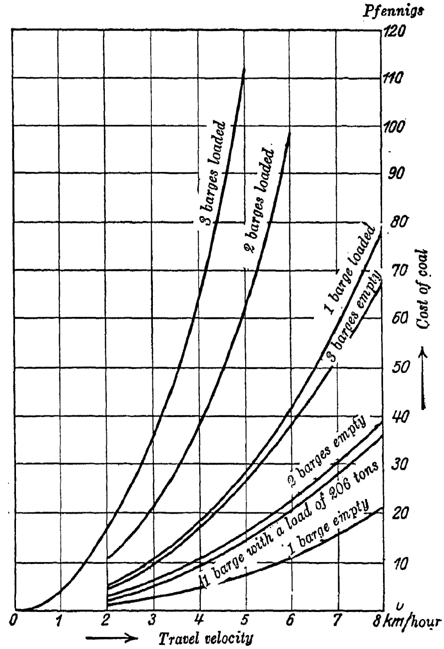


Fig. 513. Cost of coal per km. of travel for various velocities of travel

Length 65 m., breadth 8 m., draft when loaded 1.75 m. Assumed coal consumption 1.25 kilograms per HP hour. Cost of coal 18.00 marks per ton

tug with a consumption of 254 HP can tow three full barges with 1,800 tons loading at 5 km. (3.1 mi.) per hr., two loaded barges at practically 6 km. (3.7 mi.) per hr., and one loaded barge at practically 8 km. (5 mi.) per hr.; likewise with the same power, three empty barges can be towed at a rate of probably 10 km. (6.2 mi.) per hr. The fuel cost for towing has been computed, assuming that 1.25 kg. (2.75 lb.) of coal costing 18 marks per ton are required for a performance of one HP hour. The costs are indicated for various navigation velocities in Figs. 512 and 513. Fig. 512 shows fuel costs in a very general manner for a barge-train kilometer in pfennigs per hour. Fig. 513 shows the cost per barge-train kilometer; it is derived from Fig. 512 by dividing the cost per hour by the hourly transportation performance. Fig. 516

indicates the towing cost per ton-kilometer net carrying capacity. Thus, the coal cost for 100 ton-km. with a fully loaded barge rises from approximately .7 pfennig, at a velocity of 2 km. (1.2 mi.) per hr., to 3.0 pfennigs at 4 km. (2.5 mi.) per hr., and to approximately 13.1 pfennigs at 8 km. (5 mi.) per hr., the cost rising more rapidly than the square of the velocity. The coal cost for moving partially loaded barges is significantly higher per unit content than for full barges. By towing two and three barges one after another, the fuel cost per ton-kilometer increases with the number of trailers. These statements are evident from Table No. 10 which has been derived from the data in Table No. 9.

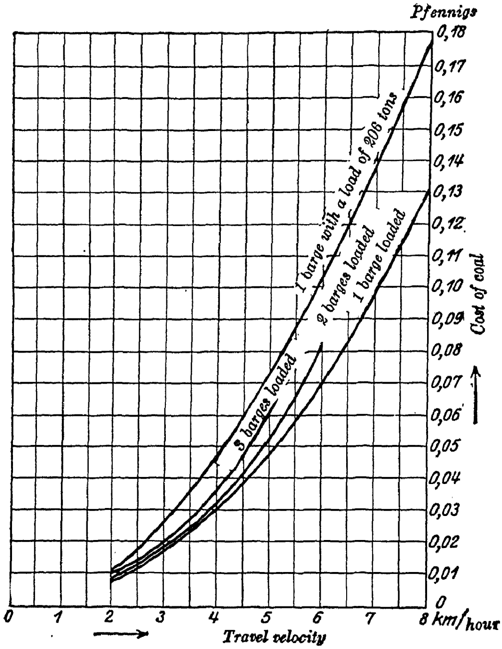


Fig. 514. Cost of coal for 1 ton km. of useful load with various rates of travel

The same information is presented in Fig. 514 for the ton-kilometer net load; for example, at 5 km. (3.1 mi.) per hr., with one trailer, the cost is .047 pfennig; with two trailers, .053 pfennig; and with three, .062 pfennig. Since the resistance of the towboat must also be overcome, this amounting to about 5 per cent for one trailer, the relative increase respectively is about .044 pfennig, .050 pfennig and .059 pfennig for one, two and three barges. This variation is due particularly to lowering of successive boats, each one coming nearer to the canal bottom and thereby raising its towing resistance.

From the standpoint of fuel consumption, it is more economical to transport goods in single ships than in barge trains. This result is just the opposite when considering the cost of providing many small towboats as compared to few large ones, since not merely the fuel cost but the total operating cost must be considered. If towing takes place by means of a towing crab (Müller system),[1] pulling of individual boats is cheaper than pulling groups.

Mattern and Buchholz made an interesting determination in that it was found from the curves compiled that, as the number of trailers

[1] Comparisons of towing systems are given later in the text.

increased, the pulling force increased in comparison with the mechanical performance. The steamboat *Clara* in developing a pulling force of 1,600 kg. (3,520 lb.) in the hawser, required

147 HP for one trailer ($v \backsim 6.5$ km. per hr.)
130 HP for two trailers ($v \backsim 4.9$ km. per hr.)
118 HP for three trailers ($v \backsim 4.0$ km. per hr.)

TABLE NO. 10

POWER REQUIRED FOR MOVING BARGES AT VARIOUS VELOCITIES

Velocity km. per hr.	Number of Barges	Empty Barges		Loaded Barges	
		Gross Load in Tons	HP	Gross Load in Tons	HP
2*	1	186	1	786	4.5
	2	2·186	1+1.5	2·786	4.5+4
	3	3·186	1+1.5+1.5	3·786	4.5+4+6.5
3*	1	186	3	786	14
	2	372	3+4.5	1572	14+14
	3	558	3+4.5+4.5	2358	14+14+20
4†	1	186	8	786	33
	2	372	8+10	1572	33+35
	3	558	8+10+12	2358	33+35+48
5†	1	186	16	786	65
	2	372	16+20	1572	65+77
	3	558	16+20+23	2358	65+77+112
6†	1	186	29	786	113
	2	372	29+32	1572	113+150

REMARKS

* Measurements uncertain for low velocities, but considerable increase with the third loaded barge.

† For higher velocities, power for every additional barge increases at higher ratio than the number of barges; in this connection no account has been taken of the fact that, according to Mattern and Buchholz, the pulling force increases more rapidly than the measured HP. Hence the increase is larger than indicated.

In this discussion the various velocities have not been taken into consideration. It may not be concluded that, because of saving of coal, it is more economical to transport with many trailers, because the increase in pulling performance is attained only at the cost of velocity. The coal cost is greatly reduced at lower velocities. A direct comparison of towing performance at various velocities, at least with vari-

ous numbers of trailers, would lead to incorrect inferences, because concluding that towing becomes cheaper by increasing the number of trailers is entirely contrary to the facts brought out in the foregoing discussion. In practice, a definite optimum velocity must be determined for a canal. Thereafter, a study must be made to determine whether it is more favorable to operate with one or several trailers.

Much experimental work is still necessary for a complete clarification of the question concerning the number of barges which should be towed. Towing operations on a canal are always considerably more expensive than on large rivers. Because of dangers of the current, the crew costs are usually larger for river navigation. The total cost of transporting coal on the Rhine from Ruhrort to Mannheim (326 km., 2,024 mi.) is one pfennig per ton-km. at low water and .3 at extreme high water (figures for 1911).

d. Navigation Methods

1. General; Drifting and Sailing

On all waterways, artificial as well as natural, ships may drift with the current, sail or be towed from the shore or with towboats. Drifting of ships with the current comes into consideration for artificial waterways if there is enough current. The velocity of the current is frequently great enough in canalized rivers and water power canals, the latter being sometimes used for navigation. When drifting with the current, for the same draft, large ships drift more rapidly than small ones.

Drifting of ships downstream is of little practical importance. Where barges are towed upstream by tugs, the tugs must also return downstream and are thereby forced to offer return service at a low rate. Consequently, many barges are also towed downstream, particularly since drifting downstream under bridges is somewhat dangerous. Sailing on rivers and artificial waterways is continually decreasing. It formerly played an important rôle but is now without economic significance.

2. Towing

Towing from the shore is one of the oldest methods of transportation by water. It consists in pulling the ship from the shore by men, animals, or machines. The method was used as early as the days of ancient Egypt where man or ass towing was used, and in a similar manner throughout the middle ages. Today the process has progressed to a high development of electric towing.

Towing from the fixed shore or a suspended line similar to the Müller system is considerably more efficient than towing with tugs. The efficiency of ship propellers is so low that a ship engine can be

replaced by a towing machine developing only a fraction of the power which is necessary for the ship. Thus a properly designed towing system brings about large reductions in transportation cost.

When being towed from the shore, the ship is directed toward the center of the waterway by means of the rudder (Fig. 515). It is recommended as far as practicable to have the towing line tied near the center of the ship rather than to the bow of the ship. At sharp bends, inasmuch as the direction of pull might be directed more sharply toward the shore, contact with the bed of the convex shore might occur. To avoid this condition, guide rollers are placed on the shore to effect a bend in the direction of the towing cable.

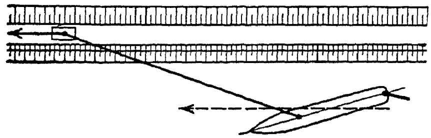


Fig. 515. Position of ship when being towed

Towing with machines on the shore can be accomplished with endless cables which move along the shore through electrical towing crabs moving on an elevated track.[1] Investigations by Block indicated that towing with electric locomotives is economical only for total annual traffic amounting to several million tons; for lower amounts towing by boats is more economical.[2]

An unbroken succession of endless cables along the shore are required for the continuous cable method of towing. Each cable operates in a definite stretch in one direction as upper cable and in the other as lower cable. The towing line of the ship is tied to the side of the cable which corresponds to the direction of navigation. This method has been used but is not of great significance.

Three systems of towing compete with each other: arrangements with electric crabs, with electric locomotives, and with motor trucks. It seems that the competition will be fought out between the electric crab system and the motor truck system. A towing layout with electric locomotives has been installed in Germany on the Teltow Canal (Berlin). This arrangement, developed by the Siemans-Schuckert-Werke, consists of low built locomotives operating on both sides of the canal, the towing cable being fastened to an arm on the locomotive. The arrangement has proved itself very satisfactory. Towing progresses at the rate of 4 km. (2.5 mi.) per hr.

A hindrance to towing from the shore is that landing quays are

[1] Sympher-Thiele-Block, *Untersuchungen über den Schiffahrtsbetrieb auf dem Rhein-Weser-Kanal,* 1907, Berlin.— Block, Die Betriebseinrichtungen des Teltow-Kanals, *E.T.Z.*, No. 82, 1906.

[2] *Z. Bauw.*, 1908.

made difficult if not altogether impossible, because towing would have to be interrupted where ships load and unload at the shore. Consequently, in canals where towing is effected from the shore, it is necessary to arrange all harbors back of the shore and to bridge the harbor mouths with light crossings for the locomotives. The towing locomotive has no particular difficulty in climbing even fairly steep ramps.

An interesting proposal by Reg.-Baurat Koss is worthy of mention. The method involves a rail, which can be moved up or down and is fastened to inclined stop arms. The rail serves as a track for pressure wheels which are driven from a small towboat. Practically the same effect is obtained with this towboat as with a locomotive towing from the shore, since the boat grasps the fixed rail and is not driven by a propeller. Very successful experiments have been made with the method but because of difficulty with the rail details, it has not been used in practice.[1]

An arrangement which has greater possibilities of development was proposed by Art. H. Müller. Müller proposes to span the entire waterway with a supporting net from which a U-shaped draft rail, a so-called "conveyance chain," is hung. On the conveyance rail runs an electrical crab which is operated from the ship or from a hanging seat. The ideal of moving individual boats is solved by this invention. The construction of a cheap supporting net is one of the main points of the invention. An organization was formed in Hamburg for testing the method and successful experiments were made.[2] The scheme contains remarkable ideas and, after further development on the basis of the successful experiments, may become a significant process for towing ships.

3. Towing Ships by Steamboat

Much has been said with reference to investigations regarding towing resistance. Towing today is done entirely with propeller steamers which are dangerous to the canal bed because of the formation of strong eddies, and to the shore because of the waves generated. Experiments by Mattern and Buchholz show that on existing canals the velocity of

[1] The method by Koss is similar to chain-draft navigation in which one chain lies on the bed of the river. The chain steamboat lifts the chain and pulls itself along, allowing the chain to fall back to the ground behind the ship. The boat going downstream must drop the chain upon the approach of a boat traveling upstream; a finder is provided so that, after passing, the boat directed downstream can again be hooked to the chain. Koss provides for raising the rail by means of gear mechanism through which the rail passes.

After canalization of the Elbe from Aussig upstream the chain method was removed, and was removed from the Spree at a still earlier date. However, it is still used in some localities, for example, on the Neckar between Mannheim and Heilbronn.

[2] *Z. Binnensch.*, 1926; Lecture by Müller.

ships may not be increased beyond 8.5 km. (5.3 mi.) per hr., if wave formation is not to endanger the shore.[1] Fig. 516 shows that with conventional towboats, a wave height of 15 cm. (5.9 in.) arises at a velocity of 8.5 km. (5.3 mi.) per hr., while an increase of velocity to 11 km. (6.8 mi.) per hr. increases the wave height to over 30 cm. (over 12 in.); at 12.5 km. (7.76 mi.) per hr., the height reaches 90 cm. (35 in.). With a velocity of 8.5 km. (5.3 mi.) per hr. no movement of the riprap at the shore could be observed. The velocity of navigation of barge trains on the Midland Canal was to be 5 km. (3.1 mi.) per hr., but for the use of the very irregular tow park it was reduced to 4 km. (2.5 mi.) per hr. Four or five trailers are pulled instead of two[2] as originally intended. On the Teltow Canal the velocity also amounts to 4 km. (2.5 mi.) per hr. Attaining larger velocities necessitates more powerful tugs. It is improbable that towing velocities as low as 4 km. (2.5 mi.) are economically correct. According to pre-war prices (Fig. 514), the coal cost was .037 pfennig per ton-km. with three trailers traveling at 4 km. (2.5 mi.) per hr. The curves indicate that for four trailers the cost would reach .045 pfennig per ton-km. Towing with two trailers at 6 km. (3.7 mi.) per hr. would make .082 pfennig per ton-km. fuel necessary and a mechanical performance about twice that required with four trailers. The circulation of ships would thereby be much better, the goods would reach their destination earlier, and the utilization of the barges and tow park would be much better. Increase of coal cost from .045 to .082 pfennig per ton-km. should not be taken as a hindrance, since this forms only a small part of the total transportation cost. For example, in the stretch Gelsenkirchen–Berlin in an official memoir, the computed operating cost was 1.1 pfennig per ton-km. of which the coal consumption was only $\frac{.037}{1.1} = 3.4$ per cent. The newly

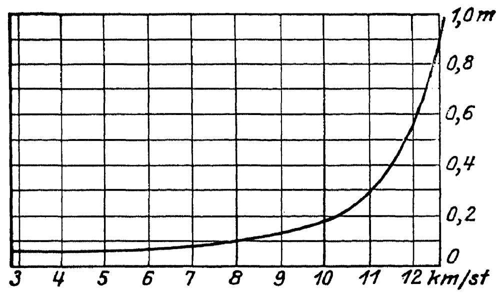


Fig. 516. Relation between velocity of travel and wave height at the shore of a canal under the influence of a single tugboat

[1] Small passenger steamships form an exception. In 1925 a high-speed motorboat was provided by the Hanover Waterway Authority, which traveled at a speed of 45 km. (28 mi.) per hr. and generated a smaller wave than the smallest motorboat hitherto possessed by the Waterway Authority.

[2] The velocity is to be again brought up to 5 km. (3.1 mi.) per hr.

planned cross-sections allow comparatively high velocities. Attacks on the bed caused by action of the propellers at the higher velocities can be greatly diminished by application of the Flamm-Buchholz slabs. However, in the Dortmund-Ems Canal the slabs caused attack of the slopes and have been discarded.

It has been observed that the effect of propellers on the bed is more pronounced the deeper the propeller lies. At present the propeller diameters range from about 1.15 to 1.2 m. (3.77 to 3.9 ft.). With these the deepest point of the propeller should not be more than about 1.4 m. (4.5 ft.) below the water surface. It has also been found that with a depth of 3 m. (10 ft.) to the bed, a mechanical performance of 90 to 100 HP with twin-propeller steamships should not be exceeded. The same holds for propeller steamships with two rudders or a rudder in front of the propeller. Single-propeller ships with the rudder in back of the propeller have proved very harmful in their attack on the bed of the canal. After these experiments, steps were taken to limit the mechanical power of ships on the Berlin-Stettin Canal, so that although larger ships were allowed to traverse the canal, the mechanical performance was limited by a special arrangement. The limiting apparatus was affixed with a lead seal at the beginning of a journey. By adopting a State tow monopoly as on the Midland Canal, there is opportunity to install particularly suitable tugs. The depth of this canal is 3.5 m. (11.5 ft.), thus making operation at higher velocities possible.

e. Computation of Freight Costs on Inland Waterways[1]

The computations indicated herein are made according to the method first used by Sympher; the newer computations by Dr. Werner Teubert for the Hansa Canal are also discussed. Differentiation is made between the ship costs which arise in maintaining ships and crews, the cost of towing, fees charged for navigating on canals and rivers, and the secondary costs for insurance, depreciation, and the like.

1. Ship Costs

Computations are made for a ship having a net capacity of 650 tons and another having a net capacity of 1,030 tons. Two hundred and ninety navigation days are assumed as a basis, considering the ship to be in use throughout this period. The ships are to be fully loaded each time on the outward trip and are to transport an average of 20 per cent capacity on the return trip. No importance need be attached as to whether the freight divides itself in some other manner between the

[1] Translator's Note: To avoid undue complication the German system of units is retained in this discussion and American equivalents are not presented.

outward and return trip, if the return journey is not computed as more than 20 per cent of full loading. One day is assumed to be required on each trip as waiting time for loading, that is, two days on a round trip. The waiting period would probably amount to more than this in the case of ships of independent ship owners. Furthermore, the period of layover is dependent on the mechanical loading and unloading capacity of the port. Three cases are here investigated, including turnover rates of 200, 300, and 500 tons per day. Small excesses are not considered; however, it is assumed, for example, that with a 200 ton per day turnover, loading a 650-ton ship requires three days and unloading four days; that is, the return journey with 20 per cent loading (130 tons) requires one day each for loading and one for unloading. Consequently, the layover time is as indicated in Table No. 11.

TABLE NO. 11

Time Consumed by Ship in Port

Ship Capacity	650 Tons			1,030 Tons			
Turnover in tons per day	200	300	500	200	300	500	
	Number of Days			Number of Days			
Loading	3	2	1	5	3	2	With full loading
Unloading	4	2	2	6	4	2	With full loading
Wait on loading	1	1	1	1	1	1	
Loading	1	1	1	1	1	1	With 20% return load
Unloading	1	1		1	1		With 20% return load
Wait on loading	1	1	1	1	1	1	
Total layover per round trip	11	8	6	15	11	7	

Table No. 12 gives information concerning these ships according to the status of 1913.

Assuming a travel rate of 60 km. per day for the average of the outward and return journey, for a total round-trip distance of n km. and an average load in each direction of $.5 \cdot \frac{120}{100} \cdot 650 = 390$ tons for the small ship and $.5 \cdot \frac{120}{100} \cdot 1030 = 618$ tons for the larger, the cost during the journey is $100 \cdot \frac{37}{60 \cdot 390} = .16$ pfennig per ton-km., and $100 \cdot \frac{48}{60 \cdot 618} =$ .13 pfennig per ton-km., respectively, independent of unloading and loading time. At a turnover performance of 200 tons per day, the lay-

over cost for the small ship, which conveys a total of $\frac{120}{100}\cdot 650=780$ tons, is $100\cdot\frac{37\cdot 11}{780}=52$ pfennig per ton; hence, for n km. per round trip, the cost is $52/n$ pfennig per ton-km.

TABLE NO. 12

DESCRIPTION OF SHIPS USED IN MAKING COST ESTIMATE FOR TRANSPORTATION OF GOODS

Principal Value for Estimate	650-Ton Capacity	1,030 Ton Capacity
Length	65 m.	80 m.
Breadth	8 m.	9.2 m.
Draft	1.75 m.	2.0 m.
Carrying capacity	650 tons	1,030 tons
Total construction cost of ship	47,000 marks	68,000 marks
Annual charges for interest, liquidation, management, maintenance, and insurance — 15%	7,050 marks	10,200 marks
Annual cost for manning	3,700 marks	3,700 marks
Total annual cost	10,750 marks	13,900 marks
Total cost of one navigation day of 290 days	$\frac{10{,}750}{290}=37$ marks per day	48 marks per day
Secondary values which are not included in the computation		
Construction cost per ton carrying capacity	72.3 marks per ton	65.7 marks per ton
Construction cost per ton per navigation day	$\frac{37\cdot 100}{650}=5.7$ pfennig per ton per day	$\frac{48\cdot 100}{1030}=4.7$ pfennig per ton per day

For the large ship with a total freightage out and back of 1,236 tons, the corresponding layover cost will be $100\cdot\frac{48\cdot 15}{1236}=59$ pfennig per ton and $59/n$ pfennig per ton-km. With a 200 ton per day turnover capacity, the combined conveyance costs for the small ship are $\left(\frac{52}{n}+.16\right)$ pfennig per ton-km.; for the large ship $\left(\frac{59}{n}+0.13\right)$ pfennig per ton-km. Computations are made in a similar manner for higher turnover rates. The shipping costs are summarized in Table No. 13.

The conveyance costs for a journey length of n km. can be computed from the figures in Table 13. For example, with the 650-ton loading and 200-ton turnover rate, the cost will be $(52+.16\cdot n)$ pfennig per ton. The cost per ton for various distances transported are given in Table No. 14.

These costs are applicable only to waterways which can be navigated with full loading the entire year. If only partial loading can be taken during part of the year, this must be properly taken into account. Likewise, in the annual cost for manning the ship, it should be observed that on some rivers, such as the Weser, one additional man is required in the crew.

TABLE NO. 13

SHIPPING COSTS

Loading and unloading rate	600-ton ship with 650-ton loading Pfennig per ton-km. shipping cost	1,000-ton ship with 1,030-ton loading Pfennig per ton-km. shipping cost
200 ton per day	$\frac{52}{n} + 0.16$	$\frac{59}{n} + 0.14$
300 ton per day	$\frac{38}{n} + 0.16$	$\frac{43}{n} + 0.13$
500 ton per day	$\frac{29}{n} + 0.16$	$\frac{28}{n} + 0.13$

TABLE NO. 14

COST OF MAINTAINING SHIP AND CREW

Length of journey out and back km.	Turnover performance in tons per day	Shipping cost in pfennig for 650-ton loading		Shipping cost in pfennig for 1,030-ton loading	
		per ton-km.	per ton	per ton-km.	per ton
300	200	0.33	100	0.32	98
	300	0.29	86	0.27	82
	500	0.26	77	0.22	67
400	200	0.29	116	0.28	111
	300	0.26	102	0.24	95
	500	0.24	93	0.20	80
500	200	0.26	132	0.25	124
	300	0.24	118	0.22	108
	500	0.22	109	0.19	93
700	200	0.23	164	0.21	150
	300	0.22	150	0.19	134
	500	0.20	141	0.17	119

Table No. 14 indicates the rate of decrease in shipping costs per unit distance with the increase in the total length of transportation distance.

2. Navigation Tolls and Towing Costs

Goods are divided into five classes on the new waterways (in Germany) for the purpose of fixing the navigation toll rates. Among other things there are included in

Class 1. Cotton, drugs, iron and steel wares, machine parts, grain, fruits, seed, fat, flour, grit, malt, rice, petroleum, and cut stone.

Class 2. Section steel and iron pipes of all types, foreign wood, jute, lead and zinc as fragments, clay wares, and raw sugar.

Class 3. Raw iron, hard wood as boards and beams, coal-tar oil, pitch, cement, and concrete wares.

Class 4. Mining timbers, logs and slab timber, lime, clay pipes, trass, cement, and concrete wares.

Class 5. Fertilizers (such as potassium salts, etc.), earth, ores, beets and roots, mortar, salt, slate, slag, natural and burned stone, brick and clinker, mineral coal, and peat.

Special class: Potassium fertilizer for German consumption.

The duty rates are given in Table No. 15.

TABLE NO. 15

Toll Rates on the Newer Canals of Germany in Pfennig per Ton-Kilometer Beginning April 1, 1927

Goods Classification	Rhine-Herne Canal	Rhine-Weser Canal with connection to Hanover in addition to branch canals such as the stretches of the Dortmund-Ems Canal belonging to this system (Dortmund to Bergeshövede-Bevergern)	Dortmund-Ems Canal below Bergeshovede (The stretch Bergeshovede-Bevergern to Emden)	Dortmund-Ems Canal for transportation between Dortmund and Emden	
				a Stretch south of Bergeshovede (Dortmund-Bergeshövede)	b Below Bergeshövede (Bergeshövede-Bevergern to Emden)
1	3	1.5	0.15	1.5	0.15
2	2.4	1.2	0.12	1.2	0.12
3	2	1	0.1	1	0.1
4	1.4	0.7	0.07	0.7	0.07
5	1	0.5	0.05	0.5	0.05
Potassium fertilizer	..	0.05	0.05	....	....

Tolls are also collected on the older waterways, but are smaller and divided into four classes.

To provide a conception of the towing rates, those formerly used on the Midland Canal (Germany) are here considered. On the Rhine-Herne Canal a rate of .18 pfennig per ton-km. was charged; on the other canals, .09 pfennig per ton-km. base charges were required to which an additional 10 per cent toll was added. Thus the towing rate was based not only upon the load but also upon the capacity of the ship. For a full

ship on the Midland Canal with Class 5 goods, the towing charge amounted to $.09+.05=.14$ pfennig per ton-km.; for a ship returning with 20 per cent load, on the other hand, since full capacity was taken into account, the towing rate on the tonnage conveyed would be $.09 \cdot \frac{100}{20}$ $+.10=.55$ pfennig per ton-km., in which the 10 per cent surcharge is figured only upon the actual load. This toll law may be judged in two ways. It may serve the purpose of rousing the shipper to look for return freight in order to avoid the relatively high tow charge of the return trip which amounts to practically as much as the outgoing trip for the entire barge loaded. This aim apparently was attained in the Midland Canal because at present much more than 20 per cent return freight is handled, the amount reaching 60 per cent and over. High towing costs may also have the effect of simply lengthening the layover periods if return freight is particularly difficult to obtain.

A different system is now used on the Midland Canal. The towing charge is computed partly on the basis of carrying capacity of the ship, but the surcharge upon the loading is such that a better division is attained.

Towing charges on rivers are largely dependent upon supply and demand. Table No. 16, which was compiled by Werner Teubert, gives a summary of these charges.

The lowest towing rates are paid for downstream traffic. These rates are .1 pfennig per ton-km. on the Rhine below Duisberg, on the Elbe below Magdeburg, and on the Oder from Breslau to Stettin. In some regions the towing rates are considerably higher; for example, 1.10 pfennig per ton-km. from Hameln to Hanover-Muenden and 1.22 pfennig per ton km. on the chain from Aussig to Dresden.

3. Miscellaneous Costs

To be considered in addition to navigation costs are harbor duties, insurance, connection freight, unloading and loading costs, and under some circumstances, depreciation of wares.

The harbor expenses at shipping and receiving harbors together amount to about .1 to .3 mark per ton. Insurance costs are about .01 to .03 pfennig per ton-km., depending upon the value of the goods. Depreciation occurs, particularly in connection with water transportation of coal, as a result of dumping the coal into the ship and unloading by various types of mechanical buckets. The depreciation may amount to from 4 to 7 per cent. Teubert estimates the loss at .5 mark per ton when very good loading and unloading equipment is used. The reloading costs for coal may be assumed as about 0.1 mark per ton for loading the

TABLE NO. 16

Average Towing Costs in Germany

From — To	At present marks per ton	Future cost marks per ton	km.	Pfennig per ton-km.	Remarks
Duisberg-Mannheim	0.85	1.00	350	0.24	
Duisberg-Rotterdam	0.20	0.22	216	0.10	
Bremen-Minden	1.60	1.20	163	0.75	Development and raising of Weser
Hameln-Münden	1.70	1.50	136	1.10	Development and raising of Weser
Minden-Hameln	0.85	0.70	68	1.00	Development and raising of Weser
Minden-Bremen	0.15	0.20	163	0.12	Development and raising of Weser
Hamburg-Magdeburg	0.80	0.95	296	0.32	Development of Elbe
Magdeburg-Dresden	1.70	1.50	271	0.55	Development of Elbe
Magdeburg-Hamburg	0.60	0.30	296	0.10	Development of Elbe
Dresden-Aussig	1.10	1.10	90	1.22	Chain tariff *
Aussig-Dresden	0.22	0.22	90	0.24	Chain tariff
Lauenburg-Lübeck	0.14	0.14	66	0.21	Towing monopoly
Parey-Berlin	0.25	0.30	125	0.24	Development of Plauer Canal
Spandau-Parey	0.20	0.25	107	0.23	Development of Plauer Canal
Hamburg-Berlin	0.95	1.10	376	0.25	
Berlin-Hohensaaten	0.25	0.25	100	0.25	Opening of Hohenzollern Canal
Berlin-Fürstenberg	0.25	0.30	110	0.27	
Stettin-Hohensaaten	0.25	0.25	79	0.31	Improvement of lower Oder
Breslau-Kosel	1.10	1.30	153	0.85	New canalization works
Breslau-Fürstenberg	0.35	0.40	300	0.13	New canalization works
Breslau-Stettin	0.45	0.50	490	0.10	New canalization works
Fürstenberg-Breslau	1.50	1.75	300	0.58	New canalization works
Kosel-Breslau	0.35	0.40	153	0.26	New canalization works
Hohensaaten-Stettin	0.10	0.12	79	0.15	

ship and .4 mark per ton for unloading from the ship to railroad cars. For other goods the loading and unloading costs may amount to about .3 to 1.2 marks per ton. The costs of unloading goods from canal ships into sea ships are usually lower than those occasioned in conveying goods from the railway to sea ships.

All costs taken together for average distances, say, of 400 km. (248 mi.), and with a loading and unloading capacity of 300 tons per day, the transportation of coal in 1,000-ton ships will require 1.2 to 1.3 pfennig per ton-km. Of this amount only about 20 per cent is pure shipping cost. In this computation, however, it has been assumed that the transportation takes place in a particular manner. At present the towing performance is usually considerably lower on the Midland Canal than the computed 60 km. (37 mi.) per day, and the layover periods are still greater than here assumed. Nevertheless, it is to be expected that after the completion of a larger network of canals many of the existing maladjustments will be overcome. It is recognized that emphasis should be laid upon reducing the toll charges and then the shipping and towing costs. A change in the latter will be possible as soon as heavier canal transportation develops.

B. DEVELOPMENT OF BEDS FOR CANALIZED RIVERS AND INLAND CANALS

a. Shape of Cross-Section

1. Canalized Rivers and Power Canals

In river canalization in general the cross-sections are retained in so far as they possess sufficient width; especially just above the dam section, the excessive depth and weak current make cross-section development unnecessary. Below the weir, on the other hand, it is often necessary to reshape the cross-section, even though a higher water level exists as a result of another weir further downstream, because inasmuch as the bed in this vicinity in its natural condition does not satisfy the requirements for ships in use, it will not usually suffice for the navigation of the larger ships which will be used after canalization. Since there is almost always a perceptible current below the weirs, the section must be made larger than would be necessary for canals simply because of the unavoidable current. The increase in size of section is advantageous to navigation and also reduces the velocity. A recommended form in general is that shown in Fig. 517, with side slopes not greater than 1:2 to 1:3. Power canals with moderate navigation must pay

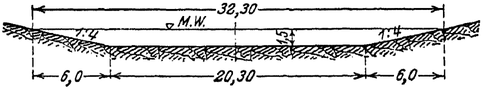


Fig. 517. Cross-section of the Aller

for themselves principally through the power gained. These canals demand a maximum economy in head so that as much energy as possible may be saved. Accordingly, in general, an average velocity of .8 m. (2.6 ft.) per sec. should not be exceeded. Direct comparison with large rivers is not possible because the cross-sections for these are usually much larger. Investigations which have been made indicate that considering the towing resistance alone as a measure of economy would be a great mistake. It is not especially important whether a tug must develop 200 or 300 HP as long as the total freightage cost by water is less than by rail. The optimum must be sought in a combination of an average size cross-section and average towing force. In power canals and canalized rivers, a value of $n = F/f = 7$ should usually not be exceeded.

A particular type of channel development of large rivers was proposed by Böhmler for canalization of the Rhine between Basel and Lake Constance. His design included a double cross-section in which on one side a flat, rough bed for upstream navigation is provided with the necessary water depth; next to it without an intermediate separation dam would lie the deeper bed of the main stream. The barge trains moving upstream would navigate over the shallow rough part of the section with diminished current, while the ships moving downstream would take the section having the stronger current. Experiments in the hydraulic laboratory of the Technical University of Berlin indicate that the desired conditions would occur. Some objection has been made to the proposal, but there seems to be no doubt that this type of development is an important advance in the design of navigable channels. It provides a

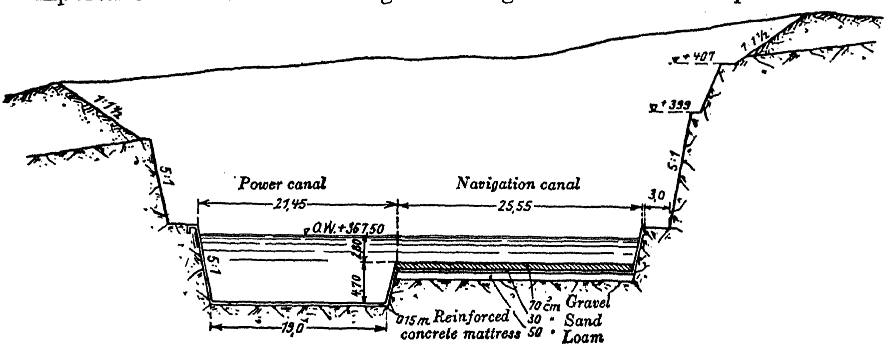


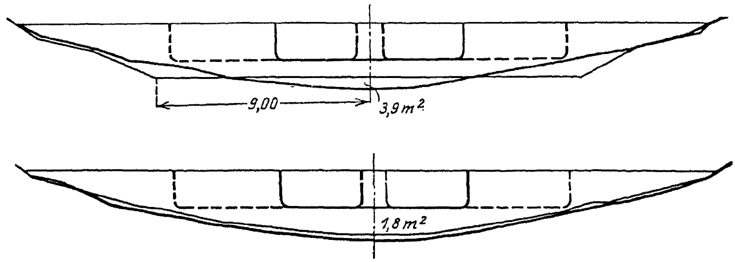
Fig. 518. Cross-section of various depths (Rhine)

systematic, suitable layout of cross-section for large canalized rivers which are to be used for water-power channels. The cross-section design is indicated in Fig. 518. The side slopes of power canals are often stabilized with concrete slabs. Slopes of canalized rivers are usually most

satisfactorily protected by riprap. The construction used for canals may also be applied to river canalization.

2. Canal Cross-Sections

The design of a canal cross-section depends upon the position taken by the bottom of loaded barges. The clear distance which must exist between barges themselves and between the side slope of the canal and the barge is fixed by experience (Fig. 507). The breadth of the canal is fixed at the elevation of the bottom of the barge. The next determina-



Figs. 519 and 520. Cross-section of the Dortmund-Ems Canal at km. 160

tion depends upon the clear depth required under the bottom of loaded barges and the form of side slope allowable in the country traversed. In bed-rock stretches practically vertical walls may be used; in sand a trapezoidal form of section having side slopes of 1:2 is suitable. The latter type of section is proposed for the spur canal to Hildesheim. The clear distance between barges is usually taken the same as the horizontal distance chosen between the shore and the barge bottom. Schmies[1] recommends the design indicated in Fig. 507.

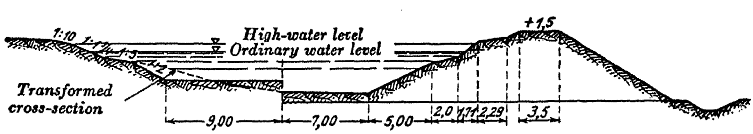


Fig. 521. Cross-section of the Dortmund-Ems Canal

Other types of cross-sections are shown in Figs. 521 to 528. One of the oldest cross-sections of the newer canals is that of the Dortmund-Ems Canal. In its original form it appeared as shown in Fig. 521. Since the depth was insufficient, the strong attack of the propeller eddies on the bottom of the canal changed its form by deepening the center as shown in Fig. 520. The eroded earth naturally deposited in the quietest part of the canal, which is along the side, thereby resulting in the forma-

[1] P. Schmies, Uber Querprofile von Binnenschiffskanälen. Doctorate Dissertation Technical University of Hanover, 1925.

tion of a trough-shape section.[1] The depth of the Dortmund-Ems Canal at usual stage was only 2.5 m. (8.2 ft.). Fig. 519 shows the dimensions of 1899 and 1905; Fig. 520, those of 1905 and 1908. Similar conditions occurred in the Oder-Spree Canal which also was of trapezoidal form with a depth of 2.8 m. (9.2 ft.) but in which the outer edge of the ship came too close to the slope (Fig. 522). By confusing the condition of insufficient depth with unfavorableness of form, it has been generally concluded that nature demands a trough cross-section. Consequently many of the

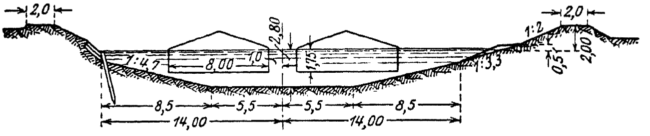


Fig. 522. Oder-Spree Canal. Eastern portion

older canals were transformed in part to trough form; a stretch of the Oder-Spree Canal, Seddinsee-Grosse Tränke, has been reconstructed as

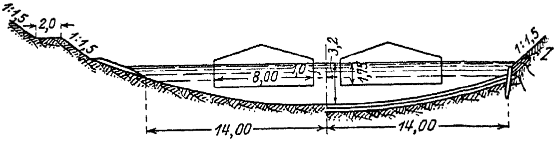


Fig. 523. Oder-Spree Canal. Western portion

a trough (Fig. 523). The Midland Canal was completely developed into the trough form; the design of the cross-section of this canal is shown in

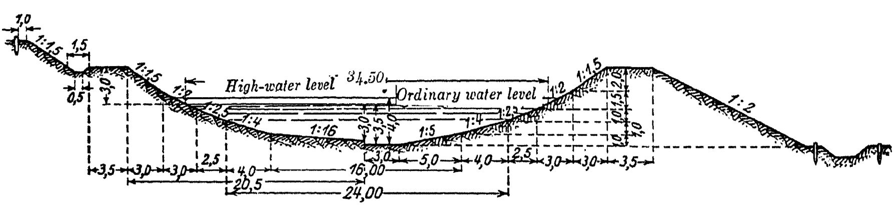


Fig. 524. Cross-section of the Ems-Weser Canal

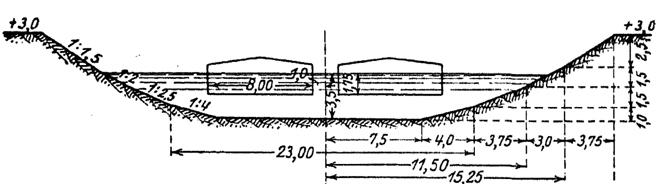


Fig. 525. Cross-section of the Rhine-Herne Canal

[1] Professor Franzius, Die Technische Ausgestaltung unserer Kanäle, 1919, p. 645, *Z.V.d.I.* Also Franzius, Die Querschnittsausbildung von Kanälen, 1926, *Z. Binnensch.*

Figs. 524 and 525. Since the trough is deep enough for the traffic which at present principally navigates in the center, in contrast to the Dortmund-Ems Canal, only small changes have occurred (Fig. 524). The correctness of the trapezoidal form is emphasized by its general use in large sea canals and also its adoption for the Rhine-Herne Canal (Fig.

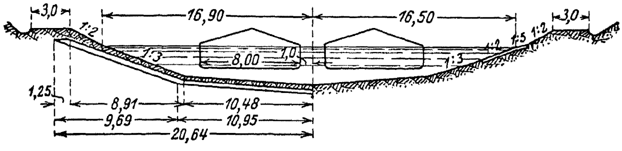


Fig. 526. Cross-section of the Berlin-Stettin Canal

525) (an extension of the Midland Canal to the Rhine) and for canalization of the Aller. Other canals having a form more nearly trapezoidal than the trough-shape are the Berlin-Teltow Canal (Fig. 527), and the

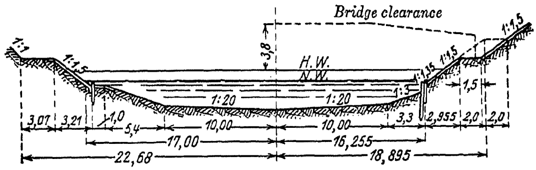
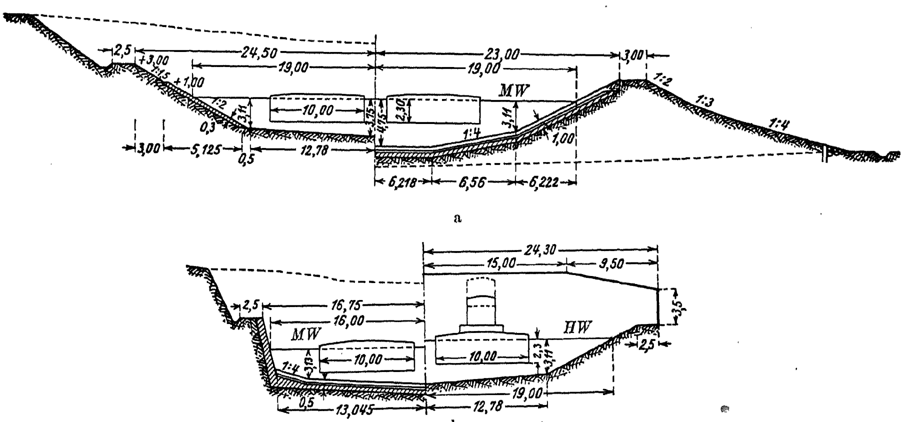


Fig. 527. Cross-section of the Teltow Canal

Elbe-Trave Canal (designed by Rehder). Typical cross-sections for the Danube-Main Canal are presented in Fig. 528; they are planned for ships of 10 m. (33 ft.) breadth and 1,200 tons capacity. It is probable



Figs. 528 a and b. Trunk line waterway between Aschaffenburg and Passau
a At points of cut and fill b At points of rock cut and under bridges

that the further use of the trough will be discontinued in future developments on the Midland Canal.

Cross-sections of sea canals are shown in Figs. 529 to 532. Fig. 529 indicates the old and new cross-sections of the Kaiser Wilhelm Canal. The former depth of 9 m. (30 ft.) was increased to 11 m. (36 ft.) but can

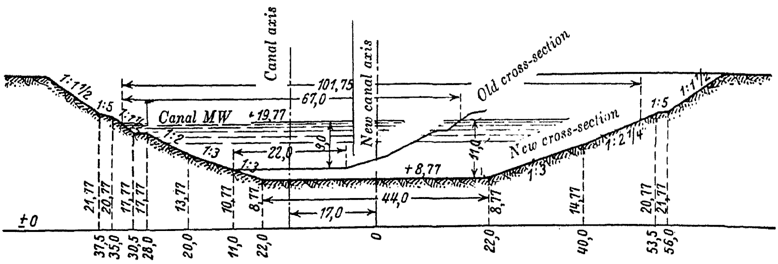
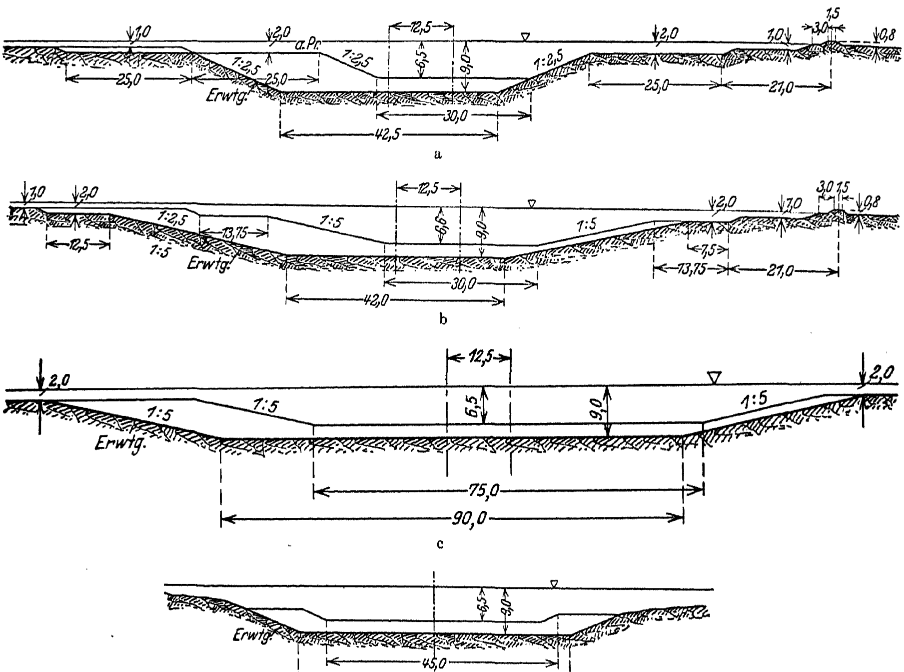


Fig. 529. Old and new cross-section of the North-Baltic Sea Canal

be still further enlarged; the original bottom breadth of 22 m. (72 ft.) was doubled. The cross-section is not large enough to allow large war



Erwtg. = Widened sections d

Figs. 530 a to d. Cross-section of the Koenigsberg Sea Canal

a Standard cross-section in sandy soil b Standard cross-section in soft silt
c Standard cross-section of the Fischhauser Wiek d Standard cross-section of the Pregel

ships to pass each other, hence recesses are constructed at the side of the canal at approximately 9 km. (6 mi.) intervals. A notable change is that the underwater berms which formerly were considered necessary are now discarded. The canal management held that the slope is more resistant without berms than with berms. It is noteworthy that a slope formerly of 1:2, now 1:2.25, extends to a depth of 5.3 m. (17.4 ft.) under water. Apparently, one might conclude that slopes of 1:2 to 1:2.5 might also be used for inland canals to a water depth of 3.5 m. (11.5 ft.).

Figs. 530 a to d show sections of the Königsberger Sea canal. Because of the small amount of excavation, a wide channel, 42.5 m. (139 ft.), could be chosen economically. The new water depth is to be 9 m. (30 ft.) and the side slopes 1:2.5 in sand and 1:5 in silt. At several points the channel has been made still wider.

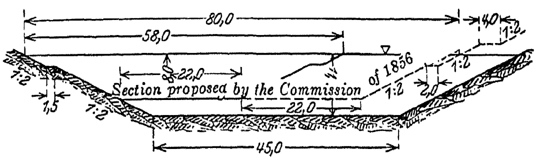
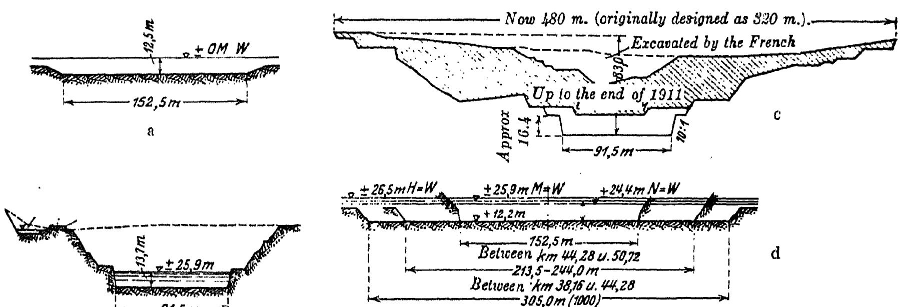


Fig. 531. Suez Canal

A typical cross-section of the Suez Canal is shown in Fig. 531. It is very similar to the Kaiser Wilhelm Canal section but possesses a water depth of 11 m. (36 ft.), bottom breadth of 45 m. (148 ft.) and side slopes of 1:2. At other locations of the canal, the slopes are 4:1 to 1:1. Figs. 532 a to d indicate sections of the Panama Canal which, corresponding to the great significance of this canal, are extremely large. The canal was largely excavated from bed



Figs. 532 a to d. Cross-section of the Panama Canal

a Open canal channel on the Atlantic side b Cross-section at the Culebra cut to Pedro Miguel. At the Culebra cut (broadened because of landslides) d Increases in bed breadth from the Culebra cut to the Gatun locks

rock; here the cross-sections have practically vertical walls, the slope being 10:1. Even at the narrowest point, the canal is wide enough to allow for two ships abreast, the bottom breadth being 91.5 m. (300 ft.)

(Fig. 532 b). The side slopes of the upper portions of the canal, partly during construction and partly subsequently because of bad landslides, were made very flat. The deepest cut amounts to 82 m. (269 ft.). For the operation of this canal it was important to provide a cross-section of 150 m. (492 ft.) breadth throughout a length of approximately 38 km. (23.6 mi.).

The velocity at which ships are allowed to travel is 15 km. (9.3 mi.) per hr. for average ships on the Kaiser Wilhelm Canal; 10 km. (6.2 mi.) per hr., for large vessels on the Suez Canal; 11 km. (6.8 mi.) per hr. in the canal and up to 28 km. (17 mi.) per hr. in the sea stretches of the Panama Canal. In general, a channel breadth of 100 m. (328 ft.) is figured for a two-ship canal, thus more than twice that for a single ship canal. This seems to be a somewhat excessive requirement; the Panama Canal, with 91.5 m. (300 ft.) breadth, may be considered as a two-ship layout in every respect. The navigation velocity allowable in sea canals is dependent entirely upon the size of cross-section. Investigations have been made by de Thierry for the Suez Canal. It was shown for this canal that a ship with 8.5 m. (28 ft.) draft, in spite of the fact that the canal cross-section was practically seven times as large as the submerged ship cross-section, when traveling at a velocity of 13 km. (8 mi.) per hr. caused a lowering of the water level amounting to 1.37 m. (4.49 ft.). De Thierry therefore recommends that the ratio $n = F/f$ should not be smaller than 5. This relation will be satisfactory for short canals, but for very long canals, carrying heavy traffic, a larger figure is desirable. For the Kaiser Wilhelm Canal with its present 827 sq. m. (8,900 sq. ft.) cross-sectional area, a ship with 25 m. (82 ft.) breadth and 8 m. (26 ft.) draft results in a ratio $n = 4.1$. Such vessels must travel correspondingly slower.

b. Bed Development and Shore Stabilization

1. Bed Development; Revetment; Concrete and Clay Waterproofing

No particular treatment has thus far been given the bed in cuts or canalized rivers. It might be considered that covering the bed with coarse, heavy, immovable material would suffice to hinder attack of the current caused by the propeller, although in most cases it is cheaper and better to make the canal bed relatively deeper, because by so doing the towing resistance is also reduced.

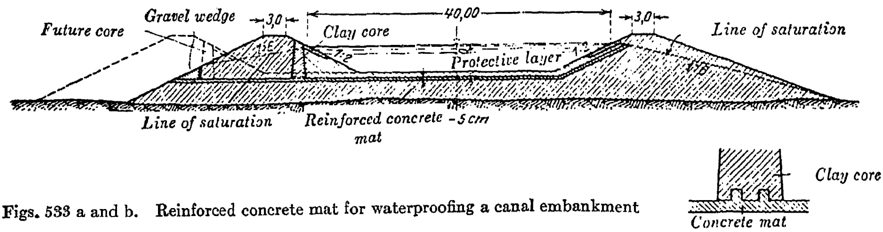
Special treatment is required where the river or canal water level lies higher than the ground-water level. In this case the water in the channel penetrates the bed and shores of the canal. The same condition occurs in canalized rivers but, in general, waterproofing the bed is here impos-

sible or so extremely difficult that it has not been done. The main difficulty encountered in making the bed water-tight in connection with canalization results from continual flow of the stream, because in order to line the river with an impervious layer the water must be diverted during construction. Penetration of the bed of a canal reaches a maximum when the deepest point of the bed lies above the ground-water surface. In general, no increase in water loss by percolation is incurred by building the canals at a still higher elevation.

The most satisfactory method of waterproofing a canal in permeable substrata consists in lining the bed with an impermeable layer. In older canals revetment set in mortar was used. The method is applicable to firm ground but is uneconomical and out of date because of the high cost of installation. Concrete layers were next adopted for this purpose. This type of construction is of importance principally at excavations through bed rock. For example, in the Marne-Saône Canal at rock cuts the vertical walls were protected by light rubble masonry and the bed was covered by a 20 cm. (8 in.) concrete layer. The canal has a depth of only 2.10 m. (6.89 ft.) and a total breadth of 12 m. (39 ft.). Compared to present waterways, it is a canal of the third order. Nevertheless, the method of covering bed rock by concrete layers may be considered good to the present time, but a protective layer of 30 to 60 cm. (12 to 24 in.) should lie over the concrete layer, as it is very likely that in the course of time the concrete layer will be broken through by ship operators.

If it is desired to use concrete for lining sand embankments, the author would propose using a reinforced concrete mat which is thin enough [3 to 5 cm. (1.2 to 2 in.)] to allow sufficient elastic deformation to follow settlement. The higher the embankment, the greater will be the settlement. Such a cover, therefore, should be constructed only after the main settlement has occurred, thus at earliest one year after completion of the embankment. Only with embankments constructed by the hydraulic process may the water-tight layer be placed immediately after completion of the structure. No importance should be attached to obtaining high flexural strength in such a reinforced concrete mat; principal emphasis should be given to toughness, that is, holding together after the occurrence of eventual breaks. Therefore, in contradistinction to other reinforced concrete structures, the reinforcing should be laid in the neutral zone. If large settlements occur, the mat may break but will become pervious only to a very slight degree, because the concrete is held together by reinforcing. Consideration of possible future enlargement of the canal may make it desirable to prolong the concrete mat at one side horizontally through the embankment, a clay core being placed in the levee extending from the mat to above the elevation of the water

surface (Fig. 533). One side of the mat then extends to the outer edge of the embankment and a gravel triangular section is constructed over it at the outer edge. When widening the canal, a trench would first be made at the bottom of the outer slope, or a concrete core would be constructed over the concrete mat. After this a new levee would be constructed next to the old one and the latter would be dredged away as far



Figs. 533 a and b. Reinforced concrete mat for waterproofing a canal embankment

as necessary. The cost of concrete mats should be compared with the cost of applying clay waterproofing. If the canal can be made water-tight by the use of clay at materially lower cost, and future extension of breadth is of no consequence, clay should be used.

A simple means of making a canal water-tight is presented when the channel is partly cut through clayey soil, but where the amount of clay excavation is not sufficient to construct complete dikes of the material; that is, when only part of the cross-section extends into the clayey ground. In such a case, if for any reason one is forced to construct sand embankments, a clay core may be built into the structure extending down as far as the clayey substratum. Complete imperviousness can be attained in this manner.

If a compact substrata can not be reached, the levee should in general be constructed of as coarse material as possible, in preference to fine material. This necessitates lining the canal completely with a clay cover. The coarse fill allows the water which penetrates the clay shell to flow away more readily without scouring out ground, than would a fill consisting of finer material. The latter allows water veins to develop, thereby causing fine soil to be scoured from the fill and finally causing a break in the levee.

Clay is the material most frequently used in Germany to make canals water-tight. Fig. 534 shows the manner in which the waterway between Berlin and Stettin was made water-tight. In particularly dangerous stretches, such as places where high embankments are used, the impervious layer was made 60 to 80 cm. (24 to 31 in.) thick and covered by a protective layer 40 to 50 cm. (16 to 20 in.) thick. At less dangerous stretches, a clay blanket 30 to 40 cm. (12 to 16 in.) thick was placed under

a similar protective layer. A clay lining such as this must be very carefully placed. It should be quite moist. It is placed in layers about 20 cm. (8 in.) thick and either tramped down by horses or rolled with a

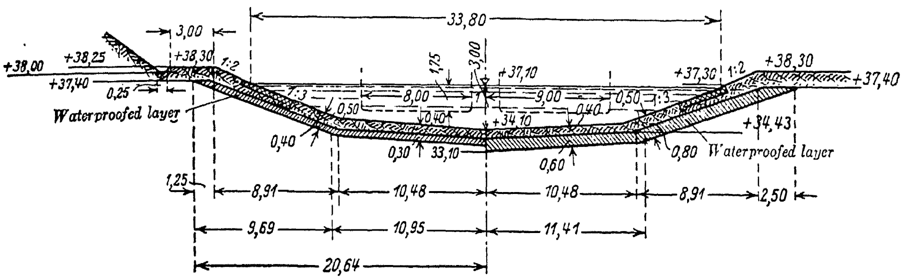


Fig. 534. Cross-section through waterproofed stretch. Trunk-line waterway from Berlin to Stettin

motor roller; rolling presents some difficulty if the clay is too soft. The protective layer is placed immediately after completion of the waterproofing blanket. The same contractor should be allowed to place both the waterproofing and protective linings, because if the work is divided, neither of the two contractors can be made responsible. An experience with a levee break on the Rhine-Hanover Canal at Dankersen verified the correctness of making the clay layer thicker on the sides. Because of the narrowness of the canal and unfavorable form (trough), the ships very frequently scraped the side slopes. Even though rammed quite tightly against the slope, the barges were again torn loose by tugs. The clayey cover was rolled off over long stretches. These damages to the slope led to excessive leakage and finally to a levee failure. Great stress should be laid upon providing a protective layer of hard fill; clayey ground softens and is not suitable.

In most cases where clay has been used for waterproofing, several years elapsed after the canal was put into operation before it became impervious. This was in part due to the fact that the waterproofing cover and protective layer were placed by different contractors. For example, in the Midland Canal between Minden and Hanover, contrary to specifications, the clay blanket was allowed to lie in the sun long enough for wide crevices to develop at about 30 to 50 cm. (12 to 20 in.) intervals. After the protective layer was placed and the canal filled with water, the clay again expanded and closed the cracks, except where prevented from doing so by penetration of the protective layer. Penetration must have occurred, because the levees leaked in places. Today the waterproofing is satisfactory.

Frequently a subsequent clay waterproofing layer becomes necessary, as in the cases of the Oder-Spree Canal, Rhine-Herne Canal, and Mid-

land Canal. Before being applied, the clay is first stirred with water to form a thick liquid. After flowing through a sieve with 1 cm. (0.394 in.) openings it is carried to the point of application through a pump hose. The liquid clay is then carried into the levee by the penetrating water. Complete imperviousness usually results within a short time. Placing of clay separately generally does not produce permanent results because the clay remains soft and is readily scoured away by propeller eddies. A gravel layer must be placed beforehand. If the depth of the canal may not be diminished, a corresponding depth must be dredged before placing the waterproofing. This was necessary on the Rhine-Herne Canal where sufficient depth was dredged to provide for a 30 to 40 cm. (12 to 16 in.) impervious layer. A protective layer of coarse gravel was then completely filled with clay. Lowering lumps of loam into the canal without stirring is not considered suitable.

If the water level of a canal lies only slightly above the ground water, considerable difficulty is encountered in construction. Under such circumstances it is necessary to lay dry not only the slopes but also the bottom of the canal. Drainage of the slopes by placing horizontal pipes into drill holes has sometimes been necessary, particularly when the ground is inclined to cave, a condition which occurred at the Hildesheim Canal. The lowering of the ground-water level during construction operations frequently results in the failure of wells at long distances from the construction site; therefore, before beginning construction of such a canal all wells in the surrounding country should be leveled in. This work should be begun long before the beginning of the actual construction work.

In case of particular types of ground, which become hard when exposed to air, it sometimes happens that the wells become so impervious at the bottom that they remain dry even after a subsequent rise of the ground-water level. Deepening of the wells is then necessary if this has not been done during the construction.

When the water level in a canal lies below the ground-water level, it is possible to hinder lowering of the ground-water level by lining the canal with clay. However, it is necessary to provide sufficient thickness of cover over the clay so that the clay lining will not be forced away by the ground-water pressure. Thus far but little use has been made of this method. It should be extensively developed in the interests of agriculture. The devastation of lands often resulting from deep canal cuts should, under all circumstances, be avoided. The designer of a canal should always keep in mind that canals are invariably developments for future centuries.

The ground-water level may be caused to rise on one side of a canal

and sink on the other side when the canal is constructed transversely to the ground-water stream and so located that it cuts into the impervious substrata at an embankment stretch. It is then necessary not only to construct clay cores but also to spread a clay lining over a thick crushed-rock layer. The crushed-rock layer forms an underground connection for the ground water from one side of the canal to the other, thereby avoiding interruption of the ground-water stream. A large number of inverted siphons would act in a similar manner but would be much too expensive. In general, a crushed rock layer of about 30 to 50 cm. (12 to 20 in.) thickness will suffice.

2. Stabilization of the Canal Shore

The severest attack on canal shores and on the shores of canalized rivers occurs slightly below the water line, because waves generated by passing ships break along the shore at this point. After close observation of the shore, Rehder developed protection for the Elbe-Trave Canal by planting a type of reed, *reth*, on the slope. The upper portion of the shore is flat like that of the Dortmund-Ems Canal. The reed was planted

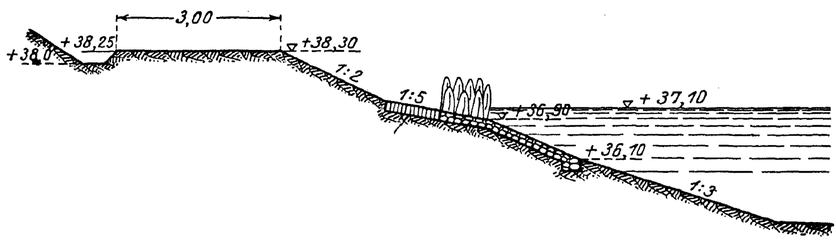
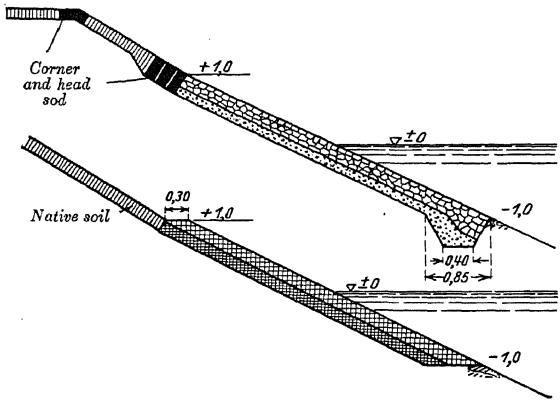


Fig. 535. Slope protection. Shrub growth along the edge of the canal between Berlin and Stettin

where the shore lies only a few hand breadths below water level, thus forming a dense green edge. The energy of the wave is so well dissipated in this "reth forest" that it becomes non-erosive. This method was used with good results on the Prussian Canal between Berlin and Stettin (Fig. 535). However, before planting the reed, a stone cover was laid extending to 1 m. (3.28 ft.) below water surface. Sod was laid above the water line. Aesthetically, the entire canal slope appears materially prettier in the landscape than the Midland Canal which can hardly be surpassed in ugliness. The shore of the Midland Canal is protected practically throughout by riprap. This riprap layer lies at a slope of 1:2 on the trough edge and extends from 1 m. (3.28 ft.) above water to 1 m. under water. Formerly, as for example on the Oder-Spree Canal, the rock was laid on top of the slope. This method was later changed so that the rock layer is embedded in the slope at many locations along the

Midland Canal. Observations disclosed that the riprap has slid, probably due to underscour; however, the error is less due to construction with riprap fill than due to trying to maintain a form of cross-section



Figs. 536 a and 537. Shore protection

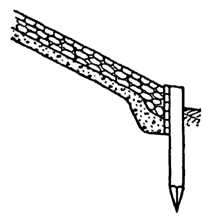


Fig. 536 b. Slope protection

which is not qualified for every type of ground. Apparently a fill similar to Fig. 537 is preferable to one like that in Fig. 536. The fill indicated in Fig. 536 a is frequently protected from sliding at the foot by short stakes driven into the bed to support a line of horizontal boards (Fig. 536 b). The slope in Fig. 537 has an advantage in that in case of sliding of the rock fill, the native soil does not follow. A slide is therefore much easier to repair than in a slope constructed as in Fig. 536 a, which generally causes much greater destruction when sliding.

A fault to be found in the use of riprap is that a canal with such protection is extremely ugly after considerable sliding of the slopes. All canals should be developed according to the pattern of the Berlin-Stettin Waterway although only narrow strips are planted with reeds. The much-used reinforced concrete slab cover, such as on the Teltow Canal, is very ugly in appearance and should not be used.

A particularly suitable material for slope protection is low-grade peat. So-called white peat has sufficient adhesiveness to form good shore protection when laid to a thickness of possibly 30 to 50 cm. (12 to 20 in.) and extending to about 1 or 1.5 m. (3.28 or 4.9 ft.) under the water level. Above the water level it is then recommended to raise a sward.

Fixing the slope of sea canals substantially need be done only near the shore; generally a sod protection will suffice. Frequently, however, it is considered suitable to lay riprap near the water line. During the first construction of the Kaiser Wilhelm Canal, large-scale experiments were made to determine the best slope protection. These investigations were

published by Fulscher[1] in 1898. The experiments were carried out on the old Eider Canal with an underwater berm lying 2 m. (6.56 ft.) under the mean water level. The experiments led to providing revetment rock with a gravel or crushed rock bedding. These interesting investigations have no particular significance for present-day cross-sections because slopes are now made without berms. In sea canals riprap is much used but should have a bedding of gravel and under this a clayey ground and also a reth growth near the top.

3. Canal Location

Unlike the former smaller canals, modern canals can not be made to follow closely the contours of the country. The elevation of a canal is mainly conditioned by the ground-water elevation. Thus in the north line of the Midland Canal between Hanover and Magdeburg, the elevation in the Drömling district (Aller, Ohre) was carefully determined (by Preussmann) after many ground-water measurements had been made, so that the canal water throughout the stretch lies a few decimeters over the ground-water surface. Such a location is necessary in country which would lose greatly in cultural value in case of drainage. In country where the ground-water level is particularly high, the position of the canal water surface might advantageously be placed below the ground-water level; thus no general rule can be given concerning the elevation of a canal with reference to surrounding country. Upon the presumption that the ground-water level averages 1.5 to 2 m. (4.9 to 7 ft.) below the ground surface, the canal water level should be approximately at the same height, because a lowering of ground water which lies so deep below the surface would be harmful to agriculture. Since the ground-water surface frequently changes its position with reference to the land elevation, it is not always possible to suit the ground-water level, because no matter how great the attempt to keep the canal course on the same elevation, experience indicates that small knolls must be pierced and small valleys closed by levees in order to lay the channel in straight lines. The fewer bends there are in a canal, the better. The line should be so located that long straight stretches will be connected by short bends. The radius of curvature of bends should not exceed 600 m. (1,968 ft.) for 600-ton ship canals and 1,000 m. (3,281 ft.) for 1,000-ton ship canals. Deviations are permissible if extraordinarily high costs arise, but then only when the bend is short. A curve with an arc of 20° and a 600 m. (1,968 ft.) radius of curvature is less inconvenient than one of 40° and 1,000 m. (3,281 ft.) radius. Sea canals, because of the relatively higher velocity of sea vessels

[1] *Z.V.D.I.* International Navigation Congress, 1898, Brussels. Third division. Third question.

and the much larger danger in the movement of giant ships, must have materially larger radii of curvature. The shortest radius of curvature in the Kaiser Wilhelm Canal is 1,800 m. (5,905 ft.). In general a radius of 2,000 m. (6,562 ft.) should be the minimum in large ship canals.

In planning the canal route, the procedure is to prepare an accurate topographic map of the country likely to be traversed, the stratification being indicated by various colors. In canal location it is invariably desired to connect large cities and raw material centers as directly as possible. By following roundabout ways, it is often possible to avoid the higher drainage divides. On the other hand, it is frequently possible to save considerable distance by directing the line over summits. An optimum must be sought and an intermediate location determined for which the annual cost of the canal together with the freight costs will produce the best solution from an economic viewpoint. Moderately high summits do not cause particular difficulty. The course should be laid in such a way that the summits traversed by the canal can be assured of feed water by gravity flow attainable from mountain or larger river districts. Up to a definite minimum, the shorter the summit stretches are, the better. The length may not be so small that large variations in level are caused by the discharge of water for locking purposes. The amount of water consumed by lockage is computed from the estimated amount of traffic. If feasible, the summit stretch is made long enough so that the variations in water level and the resulting velocities of the water in the canal do not become excessive. Otherwise the use of lift works or inclined planes is necessary to avoid feed water deficiencies.

If high ridges must be surmounted, the crossing is usually made at the deepest saddle (Fig. 562). Examples of such cuts are to be found in the Danube-Main Canal and Elbe-Oder-Danube Canal. Theoretically, piercing the ridge by means of a tunnel at the narrowest point appears advantageous, but the cost of large navigation tunnels is so great and the construction of such a tunnel under mountain large pressures so unsafe that the canal project may become endangered by such a tunnel stretch. If possible, the canal should not be laid in the side slope of river valleys, because it is then continually subjected to the danger of caving. In large canals which connect river districts and must therefore be routed over the drainage divide, cutting the canal into the slope is frequently unavoidable, but even such canals may sometimes go along tributary valleys, thereby avoiding unfavorable locations.

It is not always possible to locate a canal in such a manner that all larger cities and important transportation centers of the influence area are directly connected. Since even large cities are of no value to through traffic, it is preferable to connect them to the main line by spur canals

and to keep the main canal on a lower level. Spur canals have a further advantage in that they fit well into the street net of larger cities, thus obviating the necessity of constructing bridges which would be required if a main canal were constructed through the city.

The permissible depth of cuts and height of dikes for canals depend upon the significance of the waterway. For canals connecting cities having populations of a million or more, for example, it may be economical to make cuts 40 or 50 m. (131 or 164 ft.) deep provided the cuts are very short;[1] in connection with canals of lesser economic significance, usually long low-level roundabout routes are to be preferred. Canal embankments over 30 m. (over 98 ft.) high are not readily constructed. The Ragoser embankment on the trunk line between Berlin and Stettin, which is 26 m. (85 ft.) high, borders on the present-day maximum.

The breaks in level are located at points where there is an approximate balance of cut and fill. The balancing of earth masses, however, should not be allowed to control the design too rigidly, since throwing the material from cuts to one side and taking material for fill from borrow pits is frequently cheaper than equalizing cut and fill. The elevation of rivers to be crossed often affects the choice of the canal route. In order to facilitate spanning a river with a canal bridge, the route must frequently be displaced downstream somewhat and long, high embankments must be constructed. For example, if the MW of a river just reaches to the top edge of the levees protecting adjacent lowlands and is navigable up to a stage of 5 m. (16 ft.) over the MW, the lower edge of the canal bridge must lie at least 4 m. (13 ft.) above the highest navigable stage of the river. If the channel depth of the bridge is 3 m. (10 ft.) and the supporting structure requires a depth of 1.2 m. (3.9 ft.) below the channel bottom, the mean water surface elevation in the canal will be 13.2 m. (43.3 ft.) above the surrounding country and that of the embankment about 1.3 m. (4.3 ft.) above this. It is recommended to arrange the canal levels so that the lock lifts are practically the same. By so doing, a minimum amount of feed water is required. If one lock is constructed for a higher lift than the others, it requires a larger quantity of feed water, the excess of which does not benefit the locks which provide smaller lifts. The difficulty is moderated by the use of the thrift-lock system which makes possible more or less saving in water by variation of the number of thrift chambers. The number of thrift chambers is so chosen that the amount of loss in each of the locks is practically the same. For example, if a lock with 20 m. (66 ft.) lift has 78 per cent saving, the head loss will be 22 per cent or 4.4 m. (14.4 ft.); a lock of

[1] Cuts of 25 m. (82 ft.) depth occur seldom in inland canals. The Culebra cut of the Panama Canal, however, is 82 m. (269 ft.) deep.

15 m. (49 ft.) lift then requires a storage chamber layout in which there will be a loss of approximately 29 per cent or about 4.4 m. (14.4 ft.) head loss. Thus by proper choice of the number and breadth of the basins, satisfactory results may be obtained although the lift for successive locks be varied.

Surface-level crossings of canals over other waterways should be avoided if practicable. Canal bridges should be constructed for river crossings because traversing rivers (in addition to loss of head) requires a very different form of navigation from that on canals. Surface-level crossings of main canals should also be avoided if the canals are very heavily navigated. If the canals lie at the same elevation, however, a surface crossing might be arranged by locating the courses parallel for some distance with a corresponding increase in width. At entrances to rivers, inland canals usually must have a locked connection. This is usually the case when leveed lowlands are traversed by the waterways. If the canal assumes a higher level after passing the lowlands, it may be cut off from the lowland by continuation of the levees as far as the lift to the higher land.

At intersections with railroads it is often necessary to displace the route of the canal in order to avoid displacement of railroad stations. In general the railroad must be carried over the canal. Because of the flat road grade frequently required (1:300) for such crossings, the railroad ramps become very long and therefore expensive. For branch roads the inclination may be increased to 1:100 and for trunk lines in mountainous country this may be increased to the steepest grade occurring in the lines. Locating the railroad under the canal at intersections is ordinarily impossible except with very high embankments. The railroad may also be laid in a cut if the cut can be drained. Inasmuch as a steep ramp grade is allowable for highways (1:35 to 1:50), the latter can usually be routed over a canal without great difficulty. Two or more roads are frequently routed over a common bridge in order to provide a more economical design. Likewise, neighboring small water courses are sometimes grouped together in order to diminish the number of expensive inverted siphons.

With the exception of a few points, the fundamental principles involved in the route location of sea canals are the same as those for inland canals. With certain exceptions (such as the Suez Canal), materially larger cuts are required for sea canals, because the larger cross-sectional dimensions limit the possibility of fitting the canal to the surrounding country as readily as in the case of inland canals. Sea-level canals such as the Suez Canal need not be closed off at the ends by locks although they may be provided with such closures. The Kaiser Wilhelm

Canal is a sea-level canal which is closed at the ends. The ends of sea-level canals are closed if constructed in localities where large differences in water stage occur and where there is danger of the waterway becoming silted up. The water stage difference at the Kaiser Wilhelm Canal, for example, amounts to 8.4 m. (27.6 ft.) at the mouth of the Elbe and 5.26 m. (17.3 ft.) in the Bay of Kiel. In the Suez Canal the variation in water stage is about .8 m. (2.6 ft.) in the Mediterranean Sea at Port Said, and on the Red Sea extremity of the canal, the variation in level is about 2.5 m. (8.2 ft.). According to computations by de Thierry, the current on the Red Sea end reaches about .9 m. (2.9 ft.) per sec. The flow would doubtless have become too large on the Elbe end of the Kaiser Wilhelm Canal if locks had not been used, but on the Baltic Sea end, the current would possibly have been permissible. If at all possible, sea canals are constructed at sea-level; however, where the canal extends far inland, as in the case of the Manchester Sea Canal, a number of locks are generally necessary. When a canal which connects two seas must cross highland ridges, locks are necessary, as at Panama.

The experience gained in the construction of the Panama Canal disclosed the following fundamentals:

Long canals: Because of the differences in elevation of the ground along the best location for a long sea-level canal, so much excavation would be necessary that the breadth would have to be unduly restricted to reduce cost. In order to follow the lower stretches of land, smaller radii of curvature would be necessary than would be required by a raised canal. The saving on locks is offset by the disadvantage of diminishing the breadth of the canal. Experience shows that more accidents occur in narrow cross-sections because of the ramming of ships against the slope than in well-arranged modern locks. Hence a canal with several levels and a very broad cross-section is better than a narrow one-level canal. The time lost in passing ships through locks is compensated for by the more rapid navigation in the broad channel.

Short canals: The cost of several locks or lock flights is very significant. For example, in three-lift flights in a double layout similar to that at the Panama Canal, the cost roughly estimated may amount to about 120 or 150 million marks. Consequently, if the length of the canal is not great, a sea-level canal of considerable breadth can sometimes be constructed as cheaply as an elevated canal on the same site. The open sea-level canal is then to be preferred. Consideration should be given the possibility of a lock being made unusable for a period of months by a torpedo or air attack in case of war.[1] The operation of a canal is also

[1] Experiences during the World War showed that the danger of destruction by a torpedo is small; on the other hand, the danger of an air attack may be considered much greater.

made more expensive when locks are used. Furthermore, the cross-section of a short canal need not be seriously restricted.

The conditions at Panama were correctly evaluated by the American Government as being best suited for a lock canal with lock flights at each end for a total lift of 26 m. (85 ft.). This was done in spite of the verdict of an international commission, the European majority of which favored a sea-level canal (1906).

After the route location (on plain table sheets or other maps) has been completed, special investigations must be made. The geologic formations and ground-water conditions must be very carefully ascertained by the aid of numerous borings, and the possible methods of feeding the canal must be studied. If mining operations are in progress in the vicinity of the canal, consideration must be given the possibility of settlement of the ground surface. If gypsum or anhydrite districts or potassium layers are traversed, the occurrence of caves must be studied. Potassium districts particularly should be avoided. The presence of clay layers between rock strata may bring about considerable difficulty if not treated correctly. In the Panama Canal, for example, such formations caused the well-known extensive landslides in the Culebra Cut. Marshy districts should be avoided, if possible, regardless of the low cost of the land. If they must be traversed by the canal, earth embankments should be constructed beforehand in order to crowd out the sludge. It should not be attempted to dredge through country of this nature without levee protection, for several times the theoretical volume of the cut would have to be excavated (Teltow Canal). Canals can be located through high marshes without difficulty.

Route location for river canalization allows much less freedom than for canals. When a river has a very serpentined course in a broad valley, the larger river bends are usually cut off. When possible, locks should be located at points where weirs already exist. Frequently the existing pool level is not changed because the previously constructed weir has usually been built to the highest permissible limit. This is particularly true of rivers having low, flat shores. In rivers with steep shores on the other hand, such as the Fulda or Saale Rivers, the grouping together of older scattered flights is permissible, provided danger of inundating adjacent lands is not incurred thereby.

The partial canalization of small rivers is to be discouraged. In large rivers, on the other hand, the development of individual pools may serve to overcome cataracts, or cut-offs may serve to avoid long bends. In connection with small rivers, in general, it is preferable to construct parallel canals and develop them as power canals, rather than to canalize the river.

The profile of a canalized river is usually similar to Fig. 538. A new deeper bed must be dredged upstream from the point at which the water depth over the bed reaches a minimum. This work must be done under the express provision that no danger is involved of the river running wild

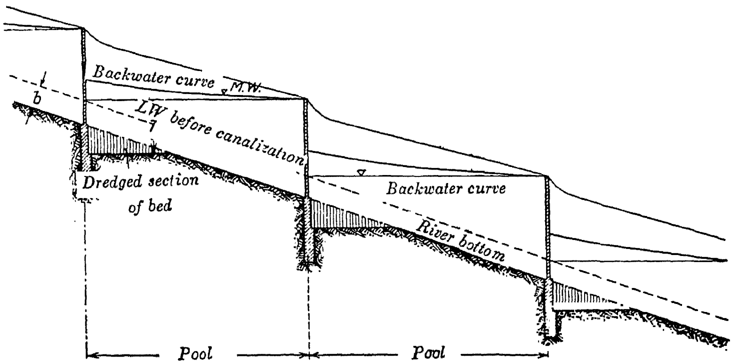


Fig. 538. Diagrammatic profile of a canalized river

in this upper portion because of too large a deposit of detritus. The detritus movement at high water, since all weirs are laid over for this stage, will not be materially varied from the condition before constructing the weirs. The river then appears as sketched in Fig. 539 in which only the fixed parts of the weir are indicated; the movable parts being laid over are not shown. With pools of sufficient length, the point B, point of break in the bed, performs in a manner similar to a weak bar in the river, the depth C as a pool. If the scouring force of the river is large enough, the location B will be eroded; because of the detritus present and the erosion at B_1 and B_2, sediment will deposit in the deep portions, C_1, C_2, etc., where the velocity is retarded. The likelihood of excessive deposits of coarse sediment depends entirely upon the nature of the river. In some rivers the amount of deposition is very small; in others, it may be great enough to make dredging section C inadvisable in the beginning. Thus, it is impracticable to recommend a scheme of canalization as indicated in Fig. 538. In large rivers subject to lively detritus transportation, the amount of dredging after very long

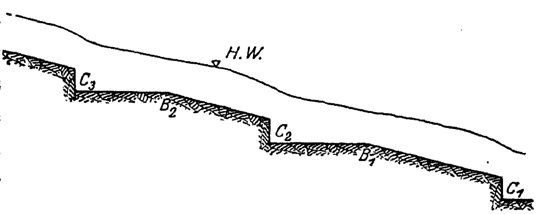
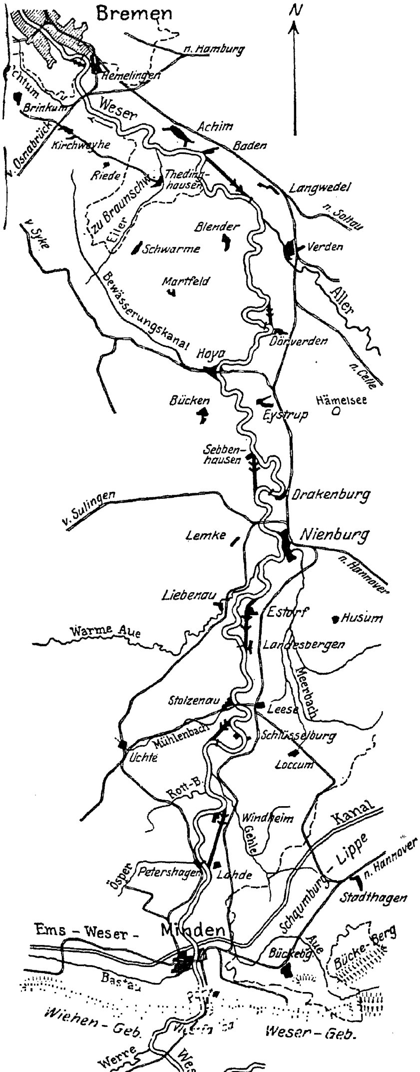


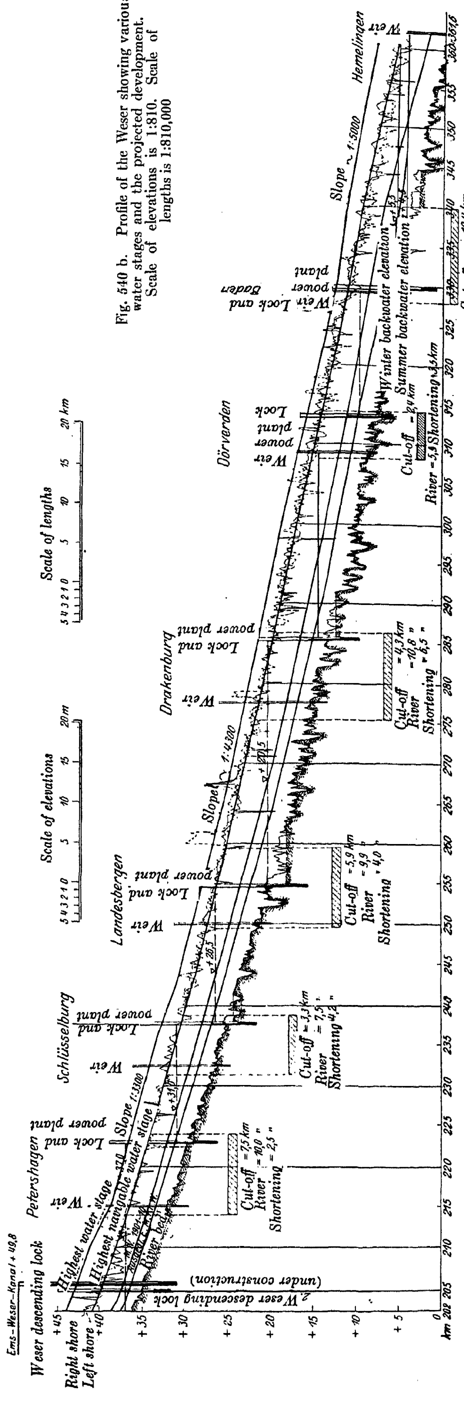
Fig. 539. Canalized river with limited depth at HW. Dredging downstream of the weir for LW navigation



Bewässerungskanal = Supply canal

Fig. 540 a. Plan of the proposed Weser canalization from Bremen to Minden

Fig. 540 b. Profile of the Weser showing various water stages and the projected development. Scale of elevations is 1:810. Scale of lengths is 1:810,000



high-water periods would become so large that the desired depth could not be attained within a year, to say nothing of the disturbance of the dredge to navigation. The rise of the backwater level, after heavy gravel deposition at C, is in all cases so small for low discharge that it need hardly be considered. The purpose of canalization and dredging is to obtain the necessary depth above B during periods of low flow. In case it is proposed to carry out canalization with dredging, very exact and extensive investigations should be made, preferably both in the field and in river hydraulics laboratories, in order to determine the feasibility of the project. If the dredged depths are small compared to the high-water depths, the possibility of development is greater than when the dredged depths are large and the high water comparatively shallow. Even in connection with large rivers, in the course of time it will become more suitable to construct parallel canals than to canalize the river. The parallel canals can be made into extensive power canals and will thereby help pay for themselves.

Figs. 540 a and b show the design for the projected Weser canalization scheme from Bremen to Minden. The locks lie in the cut-offs throughout this project, thereby providing the possibility of power development. The cut-offs must in part be prolonged by dredging in the river itself.[1] (See km. 240, 260, 290, and 340 in Fig. 540.) The total difference in elevation from Bremen (headwater at Hemelingen+4.5) to Minden (headwater+37.0) is laid out with seven lifts of from 4.5 to 6 m. (from 15 to 20 ft.). The longest cut-off has a length of 10.4 km. (6.5 mi.). Of the approximately 134 km. (83 mi.) of waterway, 33.8 km. (21 mi.) consists of cut-offs, this being about 25 per cent of the total length. The waterway will be shortened 22.3 km. (13.8 mi.), its former length being 156.5 km. (97.2 mi.).

C. WATER CONSUMPTION; SUPPLY AND DISCHARGE OF ARTIFICIAL WATERWAYS

a. Water Consumption

1. Evaporation and Seepage

The water consumption is made up of the loss in the canal by evaporation and seepage, and of the loss at locks.

The amount of loss by evaporation and seepage is dependent upon the nature of the waterway. In canalized rivers, the loss is materially greater than in canals because to the present time artificial waterproofing of the river bed has not been adopted in canalization. Since the water

[1] Prepared by Reg.-Baurat Witte, Hanover.

level is raised at a weir, the water penetrates the bed in permeable ground and flows off in the substrata as ground water.

Where the next level below the weir is deeper than the ground-water level, as is frequently the case in canalization, the water which penetrates from above the weir flows back into the river as an excess supply of ground water below the weir. Thus a portion of the percolated ground-water flows continuously toward the next level, the process repeating itself from weir to weir. Consequently the principal loss occurs in the uppermost pool of the canalized stream.

The magnitude of these losses is of significance in connection with power projects which are now developed in conjunction with the canalization of rivers for navigation. In the Aller canalization system, the water losses were so large that very harmful conditions arose on the adjacent land at the lower ends of the pools. Extensive raising of the land has become necessary. The material obtained from the upper end of the pool was found very useful for this purpose. On the Werra (power plant *Letzterheller*) the shores have been successfully raised by excavating material from the higher slopes. It is difficult to give figures concerning the quantity of water which penetrates the bed. However, it may be assumed that 10 to 100 liters per sec. per km. (4.25 to 42.5 gal. per sec. per mile) will be lost by percolation at locations where the water level is backed up several meters above ground water.

Using present day construction methods, the percolation loss in canals will be much less. Preussmann figured on a loss of 12 liters per sec. per km. (5.1 gal. per sec. per mile) in the design of the Midland Canal. Experiments on the present Rhine-Hanover Canal and Dortmund-Ems Canal indicated that frequently it is not necessary to assume penetration losses since extensive ground-water infiltration[1] occurs. If the canals are well lined with an impermeable coating, although there is no infiltration of ground water, it seems permissible to assume a penetration loss as low as 8 liters per sec. per km. (3.4 gal. per sec. per mile); in canals having a 34 m. (112 ft.) water surface breadth, 3.5 m. (11.4 ft.) maximum depth, and 2.5 m. (8.2 ft.) average depth. These figures have been taken as a basis for percolation losses on the Midland Canal for the stretch between Hanover and Magdeburg. It frequently occurs that an excess of water is available as a result of infiltration.

For evaporation Preussmann figured a rate of 4 to 11 mm. (.16 to .43 in.) per day for the first part of the Midland Canal; this corresponds to a maximum of 4 liters per sec. per km. (1.7 gal. per sec. per mile) in hot months. In the design computations of the Hanover-Magdeburg Canal, the evaporation was assumed as a total of 280 mm. (11 in.) for the

[1] Frequently this ground-water infiltration occurs at the expense of agriculture and must often be avoided in the general interest of the country.

winter months and 720 mm. (28 in.) for the summer months (from May to October), or a total annual evaporation of 1,000 mm. (39 in.). For the 34 m. (112 ft.) surface breadth this corresponds to an average evaporation rate of 1 liter per sec. per km. (.425 gal. per sec. per mile). The total water loss assumed for this canal, including both the losses due to seepage and evaporation, was 7 liters per sec. per km. (2.97 gal. per sec. per mile) for winter and 9 liters per sec. per km. (3.82 gal. per sec. per mile) for summer. Thus, for a 100 km. (62 mi.) length of canal, the loss in summer is .9 cu. m. (31.8 cu. ft.) per sec.

When large lakes are formed through the canalization of rivers, evaporation plays a considerably more important part than ordinarily; however, the evaporation does not increase directly with the area, but at a lower rate. The quantity can be approximately computed from the above figures. Evaporation in hot climates will amount to several times that for summer in northern Germany. The necessary figures must be obtained by experiment. The experiments should be made by submerging a metal basin into a water area of the region so that the loss can be measured. Experiments on land will give incorrect values.

2. Lock Losses

Before discussing the computation of water losses in locks, the possibility of feeding a canal through lock operations should be considered. The use of locks invariably causes water loss. The views presented in several articles and textbooks that the summit pool can be supplied with water by means of a special locking procedure are erroneous.

For example, it is considered that if the ships which come out of the summit pool are all loaded and empty ships enter the summit pool, there will be feeding of water to the summit pool in the case of low-lift locks. The assumption is made in this analysis, that the water consumption of the lock is smaller than the difference of the water displacement of ships passing each other in the ascending and descending trips, respectively. With this assumption, of course, at the end of each lockage more water must be contained in the summit pool than before the lockage. The error lies in neglecting two factors.

In the first place, the summit pool is concerned with locks at both ends and through traffic, so that on one end of the pool just the opposite occurrence takes place from that on the other. If, theoretically, water were gained at one end of the summit, a corresponding amount would be lost at the other end. The gain and loss would not even balance each other, but water would always be lost in the process of lockage.

Secondly, one may be concerned with a summit pool which lies at the end point of a spur canal. In this instance, there remains to be determined the origin of the full ships, since only empty ones enter the pool. The only possibility is that of loading the ships in the summit pool, a condition which is equivalent to emptying goods into the canal itself; the water must rise regardless of whether goods are dumped into the canal or the ships contained therein are loaded. Hence, it is the process of loading rather than the procedure of lockage that determines the rise in elevation of the summit level. The lockage procedure simply constitutes the loss of this excess elevation. The fact that no gain in

water can result from the lockage process is evident simply from the consideration that a ship upon passing the lock chamber, regardless of whether the ship is full or empty, produces no change in the water stage of the pool; on the other hand, the filling of the empty lock always causes lowering of the summit level.

Two kinds of water losses at locks are to be differentiated from each other; namely, that incurred as a result of leakage of the gates and valves, and that consumed in filling the chamber.

A report concerning the design of the Midland Canal (published February 1, 1920) is used as an example of the method of computing the water consumption. The losses incurred as a result of imperfect waterproofing of the gates and valves, according to experience with locks of 12 m. (39 ft.) breadth, amounts to 5 liters per sec. per m. (.4 gal. per sec. per ft.) height of lift. The annual loss for each lock therefore amounts to $.1575 \cdot H$ million cu. m. (for 31.5 million sec. in a year), where H is the lift of the lock in meters.

Locks for 1,000-ton ships are provided with an effective length of 225 m. (738 ft.) and 12 m. (39 ft.) clear breadth, thus providing a surface area of 2,800 sq. m. (30,000 sq. ft.) inclusive of gate chambers and valve shafts. For a lock lift of H meters and a consumption factor V for thrift locks (for example, with 5 thrift basins $V = .24\ H$), the annual consumption, assuming 1,000 lockages per year, would amount to $W_1 = 2800 \cdot 1000 \cdot V \cdot H = 2.8\ V \cdot H$ million cu. m. In the case of locks in spur canals of 100 m. (328 ft.) length and 12 m. (39 ft.) clear breadth, the surface area amounts to 1,250 sq. m. (13,400 sq. ft.) and the annual water consumption for 1,000 lockages would be $W_2 = 1.25\ V \cdot H$ million cu. m. Assuming a maximum of 18 lockages per day, the maximum pump performance for the large locks would be $p_1 = \frac{2800 \cdot 18}{86{,}400} V \cdot H = .5825$ $V \cdot H$ cu. m. per sec. for a single lock without leakage losses; in the case of the smaller locks with 20 lockages per day, $P_2 = .29\ V \cdot H$ cu. m. per sec. for each lock. It is assumed that with 1,000-ton ships the same number of alternatively ascending and descending lockages as successive ascending and descending lockages occur. In alternative lockages for two 1,000-ton barges, a volume equivalent to 2,000 tons plus the weight of the barge which is approximately 20 per cent of the capacity, is displaced, making a total of 2,400 tons in one cycle; in successive lockages in one direction, one-half of this amount, or 1,200 tons, is displaced, so that an average of 1,800 tons is displaced from the lock per lockage. However, consideration is taken of the fact that a large number of 600-ton barges will traverse the canal, so that the average displacement is considered as 1,500 tons per lockage. This figure is greatly increased if more return freight is transported on the canal. With

TABLE NO. 17

WATER CONSUMPTION OF THE CENTRAL LINE HANOVER-MAGDEBURG FOR 8,000 COMPLETE OPERATIONS OF THE LOCKS ANNUALLY
(12 million tons metric per year) (4,000 for theBranch Canal to Hildesheim)

Month of thirty days	Number of lockages per day		Pool + 65																	
			Lock I				Pump performance		Evaporation and seepage		Lock II				Total requirement		Feed water from the Oker and Ecker Reservoirs		Pump performance for Lock II	
	Main Canal and Branch Canal to Rothensee and Strassfurt	Branch Canal to Hildesheim	Loss through gates		Lockage water						Loss through gates		Lockage water							
			Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.	Million cu m.	Million cu. ft.	Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.
January	7	4	0.40	14.2	2.13	75.4	2.53	89.6	1.64	58.1	0.26	9.20	2.07	73.3	3.97	140.5	3.66	129.6	0.31	11.0
February	11	6	0.39	13.8	3.34	118.2	3.73	132.0	1.64	58.1	0.26	9.20	3.26	115.4	5.16	182.7	3.66	129.6	1.50	53.1
March.......	16	8	0.39	13.8	4.87	172.4	5.26	186.2	1.64	58.1	0.27	9.56	4.74	167.8	6.65	235.4	3.67	129.9	2.98	105.5
April........	24	12	0.39	13.8	7.30	258.4	7.69	272.2	1.64	58.1	0.27	9.56	7.11	251.7	9.02	319.3	3.67	129.9	5.35	189.4
May........	26	13	0.39	13.8	7.91	280.0	8.30	293.8	2.11	74.7	0.27	9.56	7.69	272.2	10.07	356.5	3.67	129.9	6.40	226.6
June........	27	14	0.39	13.8	8.21	290.6	8.60	304.4	2.11	74.7	0.27	9.56	7.98	282.5	10.36	366.7	3.67	129.9	6.69	236.8
July.........	27	14	0.39	13.8	8.21	290.6	8.60	304.4	2.11	74.7	0.27	9.56	7.98	282.5	10.36	366.7	3.67	129.9	6.69	236.8
August	26	13	0.39	13.8	7.91	280.0	8.30	293.8	2.11	74.7	0.27	9.56	7.69	272.2	10.07	356.5	3.67	129.9	6.40	226.6
September ...	27	14	0.39	13.8	8.21	290.6	8.60	304.4	2.11	74.7	0.27	9.56	7.98	282.5	10.36	366.7	3.67	129.9	6.69	236.8
October......	28	14	0.39	13.8	8.52	301.6	8.91	315.4	2.11	74.7	0.27	9.56	8.29	293.5	10.67	377.7	3.67	129.9	7.00	247.8
November ...	26	13	0.39	13.8	7.91	280.0	8.30	293.8	1.64	58.1	0.26	9.20	7.69	272.2	9.59	339.5	3.66	129.6	5.93	209.9
December....	20	10	0.40	14.2	6.08	215.2	6.48	229.4	1.64	58.1	0.26	9.20	5.92	209.6	7.82	276.8	3.66	129.6	4.16	147.3
	275	135	4.70	166.4	80.60	2,853.2	85.30	3,019.6	22.50	796.5	3.20	113.28	78.40	2,775.4	104.10	3,685.1	44.00	1,557.6	60.10	2,127.5
FOR 4,000 LOCKAGES PER YEAR (6 million tons metric per year) (2,000 for the Branch Canal to Hildesheim)																				
For the entire year.......	135	69	4.70	166.4	40.30	1,426.6	45.00	1,593.0	22.50	796.5	3.20	113.28	39.20	1,387.7	64.90	2,297.5	44.00	1,557.6	20.90	739.9

Month of thirty days	Pool + 55													
	Evaporation and seepage		Gate losses in Locks III and V		Lockage Water				Total requirement		Inflow from pool + 65		Pump performance for Lock V	
					Lock III		Lock V							
	Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.	Million cu. m.	Million cu. ft.
January	2.62	92.8	0.84	29.7	2.50	88.5	0.90	31.9	6.86	242.8	1.80	63.7	5.06	179.1
February	2.62	92.8	0.84	29.7	3.92	138.8	1.42	50.3	8.80	311.5	1.79	63.4	6.99	247.4
March.......	2.62	92.8	0.83	29.4	5.73	202.8	2.06	72.9	11.24	397.9	1.79	63.4	9.45	334.5
April........	2.62	92.8	0.83	29.4	8.58	303.7	3.08	109.0	15.11	534.9	1.79	63.4	13.32	471.5
May........	3.38	119.7	0.83	29.4	9.29	328.9	3.33	117.9	16.83	595.8	1.79	63.4	15.04	532.4
June........	3.38	119.7	0.83	29.4	9.65	341.6	3.46	122.5	17.32	613.1	1.79	63.4	15.53	549.8
July.........	3.38	119.7	0.83	29.4	9.65	341.6	3.46	122.5	17.32	613.1	1.79	63.4	15.53	549.8
August	3.38	119.7	0.83	29.4	9.29	328.9	3.33	117.9	16.83	595.8	1.79	63.4	15.04	532.4
September ...	3.38	119.7	0.83	29.4	9.65	341.6	3.46	122.5	17.32	613.1	1.79	63.4	15.53	549.8
October......	3.38	119.7	0.83	29.4	10.00	354.0	3.59	127.1	17.80	630.1	1.79	63.4	16.01	566.8
November ...	2.62	92.8	0.84	29.7	9.29	328.9	3.33	117.9	16.08	569.2	1.79	63.4	14.29	505.9
December....	2.62	92.8	0.84	29.7	7.15	253.1	2.58	91.3	13.19	466.9	1.80	63.7	11.41	403.9
	36.00	1,274.4	10.00	354.0	94.60	3,352.4	34.00	1,203.6	174.60	6,180.8	21.50	761.1	158.10	5,423.2
For the entire year.......	36.00	1,274.4	7.80	276.1	47.30	1,674.4	17.00	601.8	108.10	3,826.7	21.50	761.1	86.60	3,065.6

the conditions assumed, it follows that for an annual transportation of 12 million tons on the main canal, 8,000 lockages will be assumed. The lockages on the spur canals are figured at one-third of the above figure; that is, a quantity of freight amounting to .5 million tons computed for 1,000 lockages, with a possible figure of 4,000 lockages, might take care of two million tons.

Fig. 541 presents a summary of the consumption and supply conditions in connection with the extension of the Midland Canal along the central route.

The water requirements for individual pools are presented in Table 17, the reservoir capacity in the Oker and Eker valleys being previously fixed. The probable number of lockages for each month were taken into account in making the computation. The controlling factor is the length of the pools and the corresponding magnitude of the evaporation and percolation. With Fig. 541 and Table 17 as a basis, the water requirements for individual pools are summarized in Table 18. Table 19 presents a summary of the cost of installation and annual cost of the pumping layout; finally, the actual cost of supplying the canal with water is given in Table 20. In general, the tables are self-explanatory; Table 20 might be qualified with the statement that part of the reservoir system was paid for by the power developed and the excess was charged against the canal. The

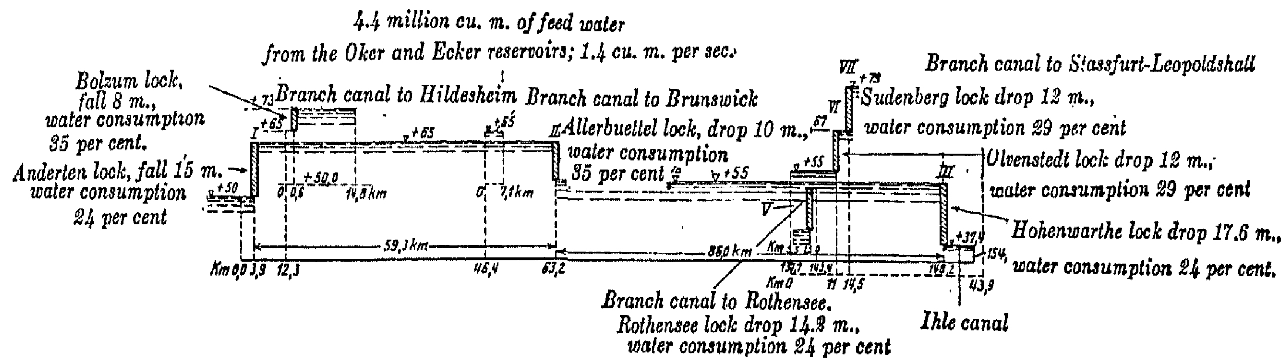


Fig. 541. Midland Canal (Germany). Profile according to an official memoir. Vertical scale 1:3000; horizontal scale 1:18,500,000

additional amount was figured at 3.3 million marks. An interest rate of 5 per cent was used as the basis for capitalization. The figures are somewhat different now inasmuch as the imperial ministry of transportation has undertaken the construction of large dams in the Bode and Oker districts, the water from which will be conducted to the Eker and Oker valleys through large pressure tunnels. This arrangement may entirely eliminate the use of pumps.

The following figures were used as a basis for determining the water requirements for the central route:

1. Evaporation and seepage.

   a. Evaporation

280	mm. in winter
720	mm. in summer
1,000	mm. for the year (for a water surface breadth of 34 m., this will amount to approximately 1 liter per sec. per km.).

b. Seepage, average, 7 liters per sec. per km.

c. Evaporation and seepage

7 liters per sec. per km. in winter

9 liters per sec. per km. in summer

The supply from the ground water and discharge from creeks was not taken into account.

2. Losses incurred as a result of leakage through the gates and valves, 5 liters per sec. per m. head (H) on the lock. The loss by leakage for each lock amounts to

$$T = .1575\ H \text{ million cu. m. annually.}$$

3. For the main canal and branch canals to Rothensee and Stassfurt, 4,000 and 8,000 lockages per year were assumed (6 million tons and 12 million tons transportation, respectively, for the main canal and 2 million tons and 4 million tons, respectively, for the branch canals); 2,000 and 4,000 lockages were assumed for the branch canal to Hildesheim (1 million tons and 2 million tons transportation, respectively).

The main canal is provided with twin locks, each of 225 m. (738 ft). length, and 12 m. (39 ft.) breadth, having a surface area of approximately 2,800 sq. m. (30,100 sq. ft.) inclusive of gate chambers and valve shafts.

The branch canals have single locks of 100 m. (328 ft.) length and 12 m. (39 ft.) breadth with an approximate surface area of 1,250 sq. m. (13,438 sq. ft.).

In the main canal for a water consumption factor m and a total lift H, the annual water consumption per lock will be

$$S_1 = 11.2\ m \cdot H \text{ million cu. m. for 4,000 lockages}$$
$$S_2 = 22.4\ m \cdot H \text{ million cu. m. for 8,000 lockages.}$$

The highest requirement (for determining the pumping capacity necessary) occurs for a maximum performance of 18 lockages per day and amounts to

$$S_{H1} = .5825\ m \cdot H \text{ cu. m. per sec.}$$
$$S_{H2} = 1.165\ m \cdot H \text{ cu. m. per sec.}$$

In the branch canals, the lockage water required annually amounts to

$$S_1 = 2.5\ m \cdot H \text{ million cu. m. for 2,000 lockages}$$
$$S_2 = 5\ m \cdot H \text{ million cu. m. for 4,000 lockages}$$
$$S_3 = 10\ m \cdot H \text{ million cu. m. for 8,000 lockages.}$$

The maximum requirement (for determining the pumping capacity) for a maximum performance of 20 lockages for each lock per day amounts to

$$S_{H1} = S_{H2} = .29\ m \cdot H \text{ cu. m. per sec. (one lock)}$$
$$S_{H3} = .58\ m \cdot H \text{ cu. m. per sec. (two locks).}$$

As already indicated, water is supplied by means of pumps or gravity flow from impounding reservoirs. Water can not be taken from the smaller water courses without sharp objection of the adjacent prop-

TABLE NO. 18

WATER REQUIREMENTS FOR INDIVIDUAL POOLS OF THE CENTRAL LINE

Water requirement of the Main Canal	8,000 Lockages					
	Normal Consumption				Maximum Consumption	
	per year		annual average			
	million cu. m.	million cu. ft.	cu. m. per sec.	cu. ft. per sec.	cu. m. per sec.	cu. ft. per sec.
(a) Pool+65						
1. Evaporation and seepage (inclusive of pool+73).......	22.5	797	0.715	25.31	0.715	25.31
2. Gate losses at Lock I.......	4.7	166	0.150	5.31	0.150	5.31
Gate losses at Lock II	3.2	113	0.100	3.54	0.100	3.54
3. Lockage water at Lock I	80.6	2,853	2.560	90.62	4.200	148.68
Lockage water at Lock II ...	78.4	2,775	2.486	88.00	4.080	144.43
(b) Pool+55						
1. Evaporation and seepage....	23.8	842	0.756	26.76	0.756	26.76
2. Gate losses at Lock III......	5.6	198	0.176	6.23	0.176	6.23
Gate losses at Lock V.......	4.4	156	0.142	5.03	0.142	5.03
3. Lockage water at Lock III ..	94.6	3,349	3.000	106.20	4.930	174.52
Lockage water at Lock V....	34.0	1,204	1.080	38.23	1.980	70.09

Water requirement of the Branch Canal to Hildesheim	4,000 Lockages					
	Normal Consumption				Maximum Consumption	
	per year		annual average			
	million cu. m.	million cu. ft.	cu. m. per sec.	cu. ft. per sec.	cu. m. per sec.	cu. ft. per sec.
1. Evaporation and seepage....	3.9	138	0.125	4.425	0.125	4.425
2. Gate losses at Lock IV......	1.3	46	0.040	1.416	0.040	1.416
3. Lockage water at Lock IV...	14.0	496	0.444	15.717	0.810	28.674

Water requirement of the Branch Canal to Stassfurt-Leopoldshall	8,000 Lockages					
	million cu. m.	million cu. ft.	cu. m. per sec.	cu. ft. per sec.	cu. m. per sec.	cu. ft. per sec.
(a) Pool+79						
1. Evaporation and seepage....	8.1	287	0.258	9.133	0.258	9.133
2. Gate losses at Lock VII.....	3.8	135	0.120	4.248	0.120	4.248
3. Lockage water at Lock VII .	34.8	1,232	1.104	39.082	2.020	71.508
(b) Pool+67						
1. Evaporation and seepage....	1.0	35.4	0.030	1.062	0.030	1.062
2. Gate losses at Lock VI......	3.8	135	0.120	4.248	0.120	4.248
3. Lockage water at Lock VI...	34.8	1,232	1.104	39.082	2.020	71.508
(c) Pool+55						
Evaporation and seepage....	3.1	110	0.097	3.434	0.097	3.434

erty owners. Consequently, it is necessary to make arrangements for compensating those with priority rights before work is begun. Water supplied by means of pumps generally requires supplementary addition

TABLE NO. 19

Basis for Determining the Original Cost and Annual Cost of the Pumping Layout of the Central Line

1. Original cost

For pumps, motors, siphons, pressure conduits, etc., and for structures and canals, each approximately 25,000 marks, or all together approximately 50,000 marks per cu. m. per sec. pumping capacity.

Pump settings having a capacity of 1.5 times the maximum requirement P_H. Accordingly, the pump installation cost $A = 1.5 \cdot P_H \cdot 50{,}000$ marks.

(The various magnitudes of lift are not taken into account because the influence is small.)

2. Annual cost.

(a) Electric current required to lift 1 million cu. m. of water a distance of 1 m. costs (0.04 mark per kilowatt hour)

$$\frac{1{,}000 \cdot 0.736 \cdot 1{,}000{,}000 \cdot 0.04}{3{,}600 \cdot 75 \cdot 0.75 \cdot 0.90} \ldots\ldots \text{ = approximately 160 marks}$$

In addition, the cost for oiling, etc. (1 mark per 1,000 kilowatt hours = 4 marks

Total, approximately 164 marks

For a pumped quantity of water P in million cu. m. and a lift H in m., the annual cost for current and oiling is $K_1 = 164 \cdot P \cdot H$ marks.

(b) Interest, liquidation, and maintenance:

Interest	(5%)	$0.05 \cdot 50{,}000 =$	2,500 marks
Liquidation	(½%)	$0.005 \cdot 50{,}000 =$	250 marks
Maintenance and renewal of machinery	(6%)	$0.06 \cdot 25{,}000 =$	1,500 marks
Maintenance and renewal of structures	(1%)	$0.01 \cdot 25{,}000 =$	250 marks

Total per cu. m. per sec. capacity = 4,500 marks

(c) Operating labor

For the pumping plant at the locks at Anderten, Allerbüttel, and Rothensee 1 machinist and 2 attendants .... 8,550 marks
(including 25% for room and board)

For the pumping plant on the branch canals to Hildesheim and Stassfurt 3 attendants .... 7,200 marks

(d) Accordingly, the annual cost for interest, liquidation, maintenance, and operation for a maximum pump water requirement P_H amounts to the following:

For the pumping plant serving the locks at Anderten, Allerbüttel, and Rothensee .... $K_2 = (1.5 \cdot P_H \cdot 4{,}500 + 8{,}500)$ marks

For the pumping plant serving the branch canals at Hildesheim and Stassfurt .... $K'_2 = (1.5 \cdot P_H \cdot 4{,}500 + 7{,}200)$ marks

(e) The total annual cost amounts to $K = K_1 + K_2$; the capitalized annual cost is $K_0 = 20 \cdot K$.

TABLE NO. 20

Determination of Cost of Feeding Canal

8,000 Lockages per year
4,000 for the Branch Canal at Hildesheim

Lock	Quantity of pump water per year p million cu. m.	Lift H m	$K_1 = 164 \cdot p \cdot H$ marks	Peak pump-water requirement P_H cu. m. per sec.	$K_2 = 1.5 P_H \cdot 4500 + 8500$ $K'_2 = 1.5 P_H \cdot 4500 + 7200$
					(See bottom of Table No. 36)
I. Anderten	85.3	15.0	210,000	4.350	$1.5 \cdot 21.330 \cdot 4500$ $+3 \cdot 8550 + 3 \cdot 7200$
II. Allerbüttel	60.1	10.0	98,700	3.495	
V. Rothensee	153.1	14.2	357,000	7.684	
IV. Branch Canal to Hildesheim	19.2	8.0	25,200	0.975	
VI. Branch Canal	47.7	12.0	95,000	2.428	
VII. Stassfurt-Leopoldshall	46.7	12.0	92,000	2.398	
			877,900	21.330	approximately 191,100 marks

$K_1 =$ 877,900 marks

Total annual cost K = 1,069,000 marks
Interest and Liquidation = 88,000 marks

$K_3 =$ 981,000 marks

Of this amount Branch Canal to Stassfurt-Leopoldshall . . . . . . = 243,400 marks
Without Branch Canal to Stassfurt-Leopoldshall . . . . . . = 737,500 marks

Capitalized pumping cost $20 \cdot K_3$ . . . . . . = 19.62 million marks
Original cost $1.5 \cdot 21.330 \cdot 50{,}000$ . . . . . . = 1.60 million marks
Oker and Ecker Reservoirs . . . . . . = 3.30 million marks

24.52 million marks

Of this amount Branch Canal to Stassfurt-Leopoldshall . . . . . . = 5.26 million marks
Excluding Branch Canal to Stassfurt-Leopoldshall . . . . . . = 19.26 million marks

from natural flow, because the consumption may be in excess of 1.5 times the normal consumption, the latter figure being that usually used for the peak capacity of the pumping installation.

The feed canals used for supplying water to a navigation canal may be very long. P. Rehder of Lübeck, for example, planned a feed canal for the Midland Canal which was over 40 km. long; the author planned one approximately 30 km. long in the Leine district. Frequently the cost of such ditches becomes so great that it is more economical to supply water by means of pumps. Inasmuch as very flat slopes must often be used (for example 1:20,000), the ditches are sometimes of large dimensions.

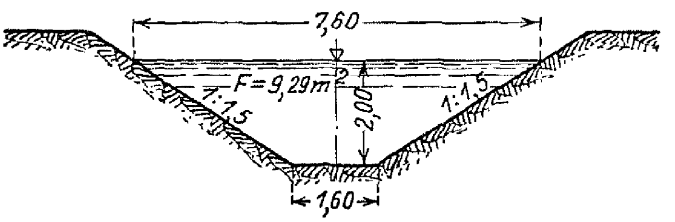


Fig. 542. Feeder canal

In his design for the central line of the Midland Canal, the author considered a feeder having a slope of 1:5,000 for 5 cu. m. per sec. This resulted in a cross-section of 2 m. depth and approximately 8m. breadth (Fig. 542).

A special inlet structure is required for feeding water to the canal; similarly, a weir and inlet structure in the river. The weir must raise the water level in the river sufficiently so that the required amount of water may be diverted at all times. The inlet structure to the diversion channel must be so arranged that the inflow can be regulated at the river up to the point of complete closure. The inlet structure at the navigation canal must provide for feeding the water to the canal without producing harmful disturbances. The most suitable place for the entrance of supply water is at a turning basin, the water being allowed to flow over as broad an area as possible to prevent harmful currents.

The manner of supplying sea canals offers no particularly unusual problems. Just as in inland canals, the water consumption is dependent upon the quantity of goods transported. Fundamentally, the sea canal does not require any more water than the inland canal if the same quantity of goods are to be transported thereon. The loss in locks may even be smaller if the lock lift is less than in the modern inland canal.

The problem of water supply was solved in an excellent manner for the Panama Canal. Here an artificial lake, Gatun Lake, of 425 sq. m. surface area forms a part of the canal and also water supply for the locks. The total lift of 26 m. (85 ft.) takes place in three steps, so that the lift per lock amounts to approximately 9 m. (30 ft.) [the maximum for low sea level is 11.9 m. (39 ft.)]. In inland canals, locks of practically twice this lift are now built. Thus far, thrift basins have not been used for sea locks. The depth of the lock chamber affects the water loss only in that it somewhat increases the loss by leakage through the gates.

b. Relief Arrangements

During periods of light ship traffic and excess water supply from the ground water or water courses emptying into the canal, there must be provision for discharging the excess supply. The relief appliance may consist of bottom outlets, spillways, or siphons. Since in the case of bottom outlets the pipe line is under a head of 2 to 3 m. (7 to 10 ft.), velocities of over 4 m. (over 13 ft.) per sec. occur. Consequently, large cross-sectional areas are not required for outlets of this nature. A pipe of 1 m. (3.28 ft.) inside diameter would then discharge about 3 cu. m. (106 cu. ft.) per sec.

The fundamental hydraulic principles for overflow outlets are the same as those for weirs. An overflow structure of concrete is built into the canal embankment. A particularly suitable relief outlet consists of the siphon, such as the patented system of Heyn, Stettin. A greater discharge capacity is possible by means of a siphon than by means of an overflow spillway of the same cross-section. Inasmuch as the water consumption of various successive locks is never equal, overflow outlets are necessary for the various locks below the upper lock.

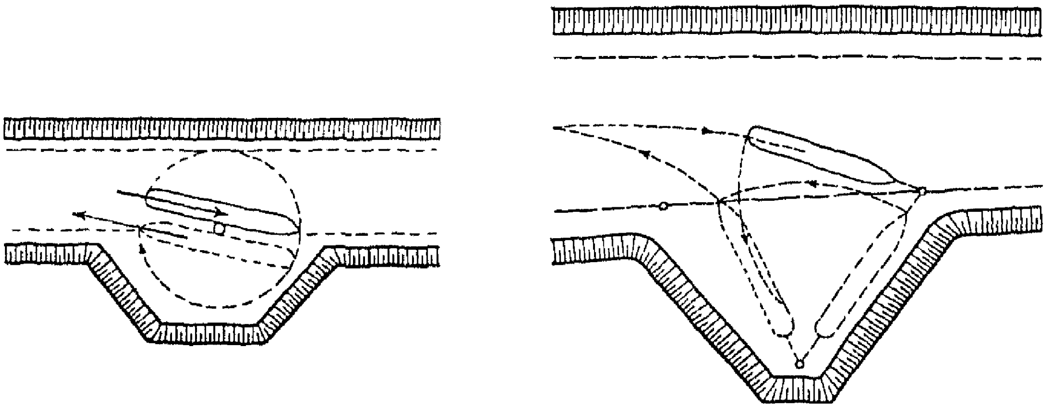
D. SPECIAL CANAL STRUCTURES

a. Lock Entrances, Shunting Places, Turning Places, Guide Works, Dolphins, and Mooring Posts

The quality of a lock depends largely upon its fore harbors and guide works. The fore harbors of the lock at Anderten on the Midland Canal are shown in Fig. 339. Space should be allowed for ships to await entrance to the lock at a point considerably distant from the lock. No difficulty is provided in arranging the guide works below the lower bay. Above the head-bay, however, care must be taken not to impair the watertightness of the bed by driving piles into it; heavy columns must frequently be set on top of the bed. An arrangement of this nature was effected at Minden. A good type of construction consists in the use of reinforced concrete caissons which are floated into place and sunk by filling them with sand. These may be stiffened by braces on the landward side. The dolphins and mooring posts at the entrance to locks do not differ from those ordinarily used in connection with harbors.

There must be turning places for ships at various points along the canal. Most canals are too narrow to allow ships to turn within the normal canal width. A turning place is provided by forming a bay at one side of the canal sufficiently large to allow all ships traversing the canal to turn. A breadth somewhat greater than the length of the ship will suffice. The ship is fixed to the shore at one side and turns about

this point. In case of canals having a large amount of traffic, the bay should extend landward far enough so that the ship will not obstruct traffic while turning. A turning place may be of the form of a half circle or of an equilateral triangle (Figs. 543 and 544). Turning points



Figs. 543 and 544. Turning basins in canals

of this nature are also required in sea canals. Turning basins are provided at intervals of 15 to 27 km. (9 to 17 mi.) along the Kaiser Wilhelm Canal; they are of semicircular shape extending 300 m. (984 ft.) landward beyond the edge of the canal.

Shunting places are necessary in all single ship (one-way) canals and rivers which are canalized for a single line of traffic. A shunting place consists of a widening of the canal sufficiently large so that one ship may pass another which is anchored temporarily. In case of a ship breadth of 10.5 m. (34.4 ft.), it is advisable to provide shunting places for barge trains of two barges, allowing a length of 250 m. (820 ft.) and breadth of 15 m. (49 ft.). Such widenings of the waterway are also used in double ship canals so that one or two ships can be anchored at the side. In large single-ship canals, such as the Kaiser Wilhelm Canal, shunting places of 1,100 m. (3,608 ft.) length are arranged at intervals of 5 to 14 km. (3 to 9 mi.). The inequality of the length of intervals between shunting places is due to the nature of the cut, the locations being chosen in low territory.

b. Culverts and Inverted Siphons

It is permissible to conduct water from a water course only if the property owner downstream of the diversion does not object. In most cases this is not true. If the water course transports considerable silt, the water must enter a settling basin before being diverted into the canal in order to prevent the canal from becoming silted up. Water courses having high potassium content and other impurities, even though the

water is very clear, should not be diverted into the canal because municipalities value the possibility of using the canal water for bathing and also because seepage from the canal would endanger the fertility of the lowlands in case the water contained certain impurities. If the canal is situated on a high embankment so that the bottom is at some distance above the surrounding territory, culverts are ordinarily used for the purpose of passing small streams through the embankment. They should be made as accessible as possible. Reinforced concrete pipes or iron pipes may be used, but the inside diameter should not be less than 80 cm. (32 in.).

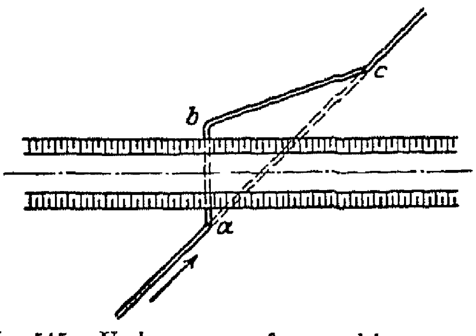


Fig. 545. Underpassage of a canal by means of an inverted siphon a — b

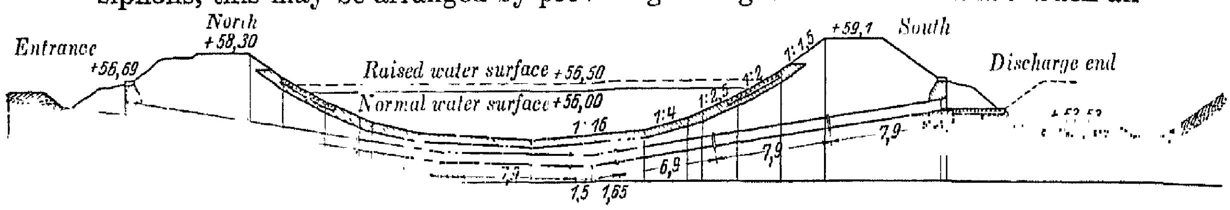
If the surrounding territory lies too high with respect to the canal, water courses which may not be severed must be conducted under the canal by means of inverted siphons. The siphon acts as a pressure conduit and requires a certain amount of head to cause the water to be forced through it. Backwater can be avoided if it is possible to lower the tail-water level by broadening and deepening the brook. If this is not possible, the upstream portion of the water course should at any rate be conducted toward the inverted siphon in such a way that no lengthening is incurred. A position of the siphon similar to Fig. 545 is suitable. It is preferable to follow the course *abc* ; however, because of the ground which might otherwise have to be purchased, it is frequently necessary to follow the line *ac*. The siphon should have a minimum inside diameter of 80 cm. (32 in.) in order to make it possible to at least crawl through the conduit. This was set as the minimum diameter for culverts and inverted siphons for the central canal route between Hanover and Magdeburg.

Inverted siphons may be constructed as arches, reinforced concrete pipes, or iron pipes. The structures must be designed for both internal hydrostatic pressures and pressures incurred as the result of the superimposed load. They should be impervious to flow from the inside to the outside and also in the opposite direction in order that the canal water will not be provided with outlets during the hot summer months when the siphons are empty. Inverted siphons must also be designed to withstand hydrostatic uplift when they are empty in order to keep the structures from being forced through the bottom of the canal in case there is but little earth cover over the siphons. An accident of this sort might result in putting the canal out of commission.

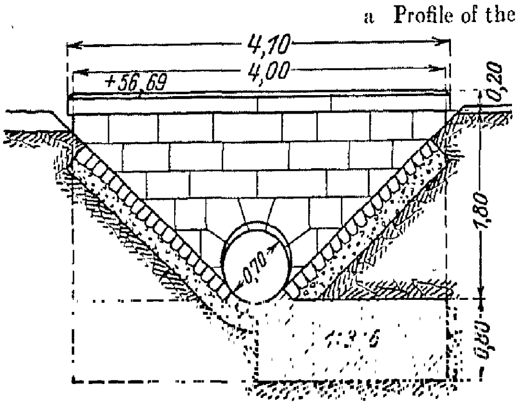
The diameter of the siphon is computed by the customary pipe

formulas with the assumption of a permissible head for maximum discharge. Although fairly high velocities are permissible for very short periods of discharge, velocity for rates of discharge which persist for comparatively long periods should not exceed 3 m. (10 ft.) per sec.

A grit chamber should be arranged at the entrance of the inverted siphon in order to make clogging of the pipe line at its lowest point less probable. Provision should be made for flushing inverted siphons. In the case of important siphons, this may be arranged by providing inlet gates from the canal. Such an



a Profile of the underpassage

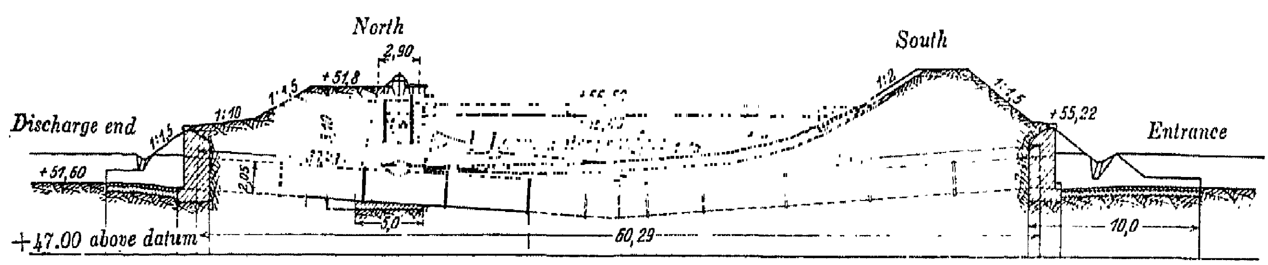


b Entrance

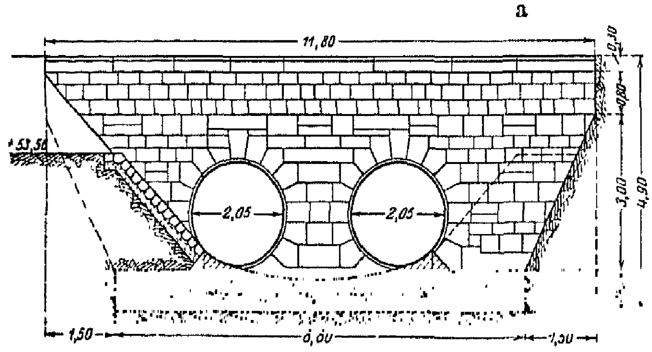
Figs. 546 a and b. Inverted siphon at Schulze-Wedeling

arrangement may also serve the purpose of an overflow for the canal. The siphon should be provided with a chain from end to end, which can be used to clear a path large enough to start the flow in case the conduit becomes completely clogged.

Formerly inverted siphons consisted of straight pipes connecting two drop shafts which served also as grit chambers. These have been superseded by structures of the type shown in Figs. 546 and 547.



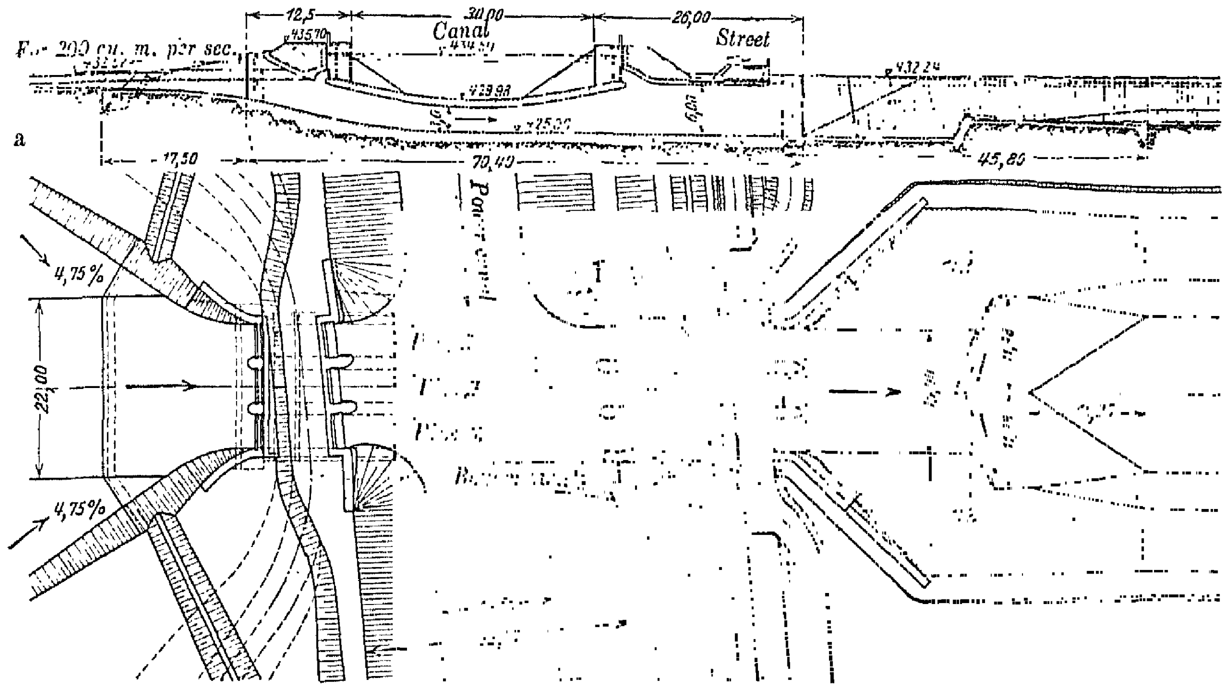
a



b

Figs. 547 a and b. Beverbach inverted siphon
a Profile b Entrance

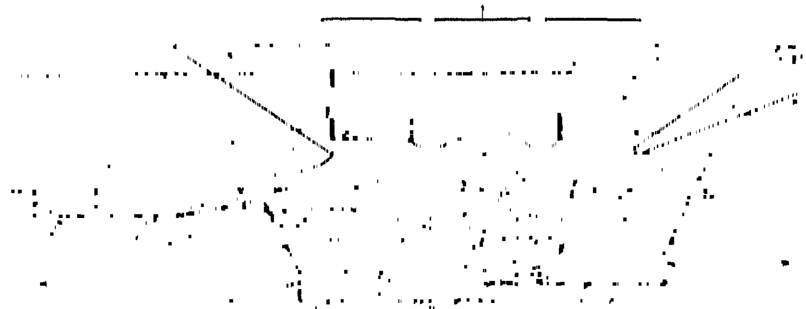
Figs. 546 a and b indicate a type of siphon for use in territory which is comparatively high with respect to the canal. The fore wall at the inlet is deep enough so that the water level is always above the top edge of



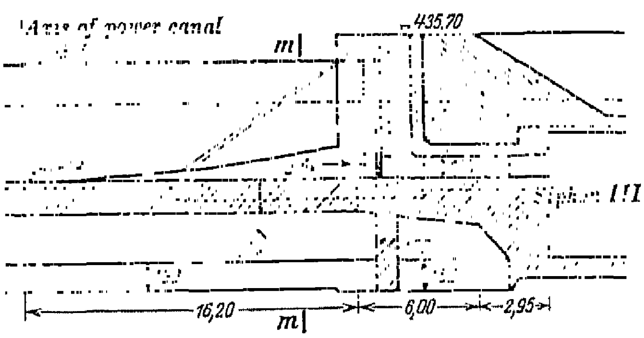
b Scale 1 : 1300.

Figs. 548 a to d. Sempt inverted flood siphon

a Ground plan b Profile
c Cross-section through siphon; at the left through the center of the canal, at the right through the entrance
d Profile through the siphon bottom outlet



c To the right of inverted siphon III Scale 1 : 500.



d

the inlet. Such an arrangement prevents floating debris from entering the culvert, the debris being removed from time to time. Figs. 547 a and b indicate an inverted siphon in which an inlet is arranged for drawing water from the canal. Other inverted siphons are shown in Figs. 548 a and b. They indicate the under passage constructed for the Sempt Creek under the large power canal [30 m. (98 ft.) breadth] of the Middle Isar (Germany). The siphon is constructed entirely of reinforced concrete. It consists of three rectangular conduits, each having a cross-sectional area of 12 sq. m. (130 sq. ft.) at the center of the canal; the cross-sectional area at both ends of each conduit is 16 sq. m. (172 sq. ft.). Gates are provided between the canal and siphon

at both sides of the canal; these have been arranged for scouring the siphon and discharging excess water from the canal.

c. Emergency Gates

Canal pools which are provided with embankments and canal bridges must be protected at the ends by gates. Any type of lock gate may be used for this purpose, but the gates are nowadays required to serve for holding water back from either side. Consequently, only segment gates and lift gates are practical for this purpose. In the newer canals of Germany only these two types have been used. Emergency gates of the segment type were used on the Dortmund-Ems Canal, then on the Berlin-Stettin Canal; more recently on the Midland Canal lift gates were installed.

The use of automatic gates was considered, but they were not adopted. In order to do this it would have been necessary to allow considerable change in stage before the gate went into operation (as would occur in the case of a break in the embankment) or the gate would have had to be put into operation by the effect of the strong current which would arise at the time of a break. Within certain limits either of these conditions might be generated by backwater caused by wind or by the passage of ships. In the case of automatic operation, there is also the danger of a barge train being caught under a gate at the time of a break. Under such circumstances either the ship might be sunk at this location or the gate damaged.

It should be possible to operate these gates simply by actuating a lever which allows them to close instantly under the influence of their own weight. In the case of an embankment failure at Dankersen the emergency gates proved satisfactory; if it had not been possible to close the gates rapidly, considerable loss of life and property might have resulted.

d. Highway, Railroad, and Canal Bridges

1. Highway and Railroad Bridges

Highway and railroad bridges over inland canals should never be constructed as arch bridges but as girder bridges or arch bridges with horizontal tension rods, so that it will be possible to raise the bridge subsequently if necessary. In the case of the Rhine-Herne Canal, because of settlement resulting from mining operations, the bridges were arranged so that they could be raised hydraulically as the ground settled, the height of the masonry foundation being built higher from time to time. Such arrangement was not provided in connection with the Midland Canal between Bevergern and Hanover, and a large number of reinforced concrete arch bridges were constructed, the raising of which

is practically impossible. It is not entirely improbable that it will be desired to raise the water level of the Midland Canal some time in the future in order to facilitate transportation. Such a measure would necessitate the reconstruction of a large number of arch bridges.

The design of highway and railroad bridges is a subject of bridge engineering, and is therefore not discussed in detail here. It is permissible to narrow a canal at bridge crossings to an extent not greater than the useful breadth of the canal, so that ships may pass under the bridge without changing their direction. According to the rule previously mentioned, the clear breadth of the canal between abutments should be at least 30 m. (98 ft.) (for ships of 10.5 m. breadth); to this distance must be added the breadth of the towing way so that the clear breadth including 3 m. (9.8 ft.) for towing ways on both sides would amount to 36 m. (118 ft.) for highway bridges and other obstructions.

The clear breadth of bridges over the Midland Canal amounts to 41 m. (134 ft.); thus the canal was not limited to its present useful breadth. This arrangement is advantageous because it makes possible subsequent broadening of the canal without lengthening of the bridges. Highway bridges of the second order should have a breadth of 4.5 m. (15 ft.) and a roadway of 3.5 m. (11 ft.); the breadth of other bridges should correspond to the importance of the bridge, provision being made for two or more lines of traffic and adequate breadth for walk ways. In cities, bridges of practically all breadths are constructed, depending upon the importance of the particular highway. The clear height of the bridge should be at least 4 m. (13 ft.), preferably 4.5 m. (15 ft.), over the highest water stage.

In the case of sea canals, highway crossings may be provided either by ferries, swing bridges, or elevated bridges. The structures over the Kaiser Wilhelm Canal are particularly well planned. The lower edges of the elevated bridges over the canal are 42 m. (138 ft.) above the highest water stage.

2. Canal Bridges

Canal bridges serve the purpose of carrying the canal over deep river valleys; they are frequently necessary in canals of the first order. When used for river crossings they are constructed to such a height that ships may navigate below them during ordinary stages of the river, and are also high enough so that they do not obstruct the river during flood flow. There is always some question as to whether a one-ship or two-ship canal bridge should be constructed. Under all circumstances every river bridging should consist of two one-ship canal bridges. In this way twofold safety is obtained and the bridges are easily repaired

and maintained since one bridge may be taken out of service during times of little traffic. Resolving the bridge into two narrow bridges also reduces the length of the transverse girders. Furthermore, one-way (one ship) bridges may be made narrower than half the breadth of two-way bridges. For example, two bridges of 12 m. (39 ft.) clear breadth are cheaper and safer than a single one of 30 m. (98 ft.) clear breadth; there is less danger of ships ramming into each other or into the side of the bridge in the case of one-way bridges than in the two-way bridges.

The increase in towing resistance forces the navigator to travel slowly over canal bridges, but in the case of short bridges, the time lost thereby is insignificant. It is assumed that tows which travel in the open canal at a rate of 4 km. (2.5 mi.) an hour must reduce their speed to 3 km. (1.9 mi.) an hour in going over the bridge. The velocity must be reduced before reaching the bridge in order to avoid the ramming together of the barges. In the case of the Magdeburg canal bridge, assuming the distance over which the velocity is reduced to 3 km. (1.9 mi.) an hour is 1 km. (.621 mi.), the time lost in crossing the bridge would amount to $20-15=5$ minutes which corresponds to $\frac{1}{3}$ km. additional length of canal.

Canal bridges may be constructed of stone, concrete, reinforced concrete, or steel. It is suitable to construct short canal bridges of reinforced concrete; the latter is also used for canal bridges having a span length of as much as 50 m. (164 ft.). For steel bridges it is recommended to use high strength steel. This material is also used for short span bridges. The clear span of canal bridges of the Midland Canal at Minden and Seelze is 24 m. (79 ft.) and the maximum water depth in the trough amounts to 3 m. (10 ft.), thus the bridge loading is extremely high.

The actual construction of the trough for canal bridges is not a particularly difficult problem. The types of construction used for other bridges of a similar nature might also be used for canal bridges. The connection of the bridge trough to the land section of the canal and the transitions over the piers, however, are important details. In the case of concrete and reinforced concrete bridges the trough is made of concrete. Bituminous waterproofing must be used and the entire inside of the trough covered with a lead skin. The inside of the concrete trough is first covered with heavy tar paper and on top of this on both the bottom and side walls a layer of sheet lead 2 mm. (.08 in.) thick on the bottom and 3 mm. (.12 in.) on the side walls. On the top of this is placed another layer of heavy tar paper followed by a 12 cm. (4.7 in.) layer of loam on the bottom. Superimposed over this is the actual protective layer for the lead lining, this layer consisting of reinforced concrete slabs. The lead lining at the sides of the bridge is protected by wooden planking. The bridge joints are capped with overhanging sheet metal, allowing the small amount of movement of the bridge which results from temperature changes and the like. The transition to the landward bearing is arranged preferably by making the pier of concrete, the upper surface being

protected and sealed by loam. The entire arrangement must be designed in such a manner that a slight amount of movement may take place between the end of the canal bridge and the supporting pier without destroying the water-seal between the two. It is questionable whether the extremely expensive lead lining is necessary. It is now possible to make concrete waterproof without adding a surface cover. The use of gunite on the surface provides further protection. Inasmuch as bridges are ordinarily put into operation long after the concrete has been poured, concrete is not likely to be disintegrated by the canal water unless the latter contains harmful impurities.

Steel canal bridges are constructed in such a manner that the trough is movable within the supporting structure, either being hung from or laid in the latter. It is required that the supporting structure be independent of the trough so that each may undergo small deformations independently. It is to be recommended, in case there is sufficient clearance available below the bridge, to support the trough on a large number of longitudinal girders, as heavy highway bridges are frequently built. An example of such a design is the canal bridge over the Leine River at Seelze (Fig. 549), which was constructed by the firm Louis Eilers of Hanover. In this structure the trough is supported on a large number of main girders and lies movable against vertical transverse girders at the sides. The clear breadth of this structure is 24 m. (79 ft.).

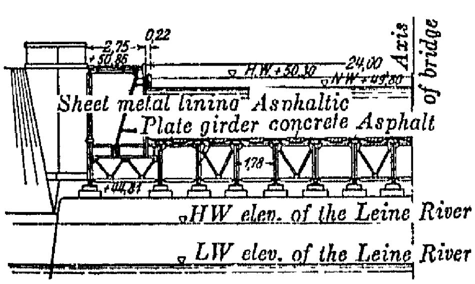


Fig. 549. Canal bridge over the Leine River at Seelze. Constructed by L. Eilers of Hanover

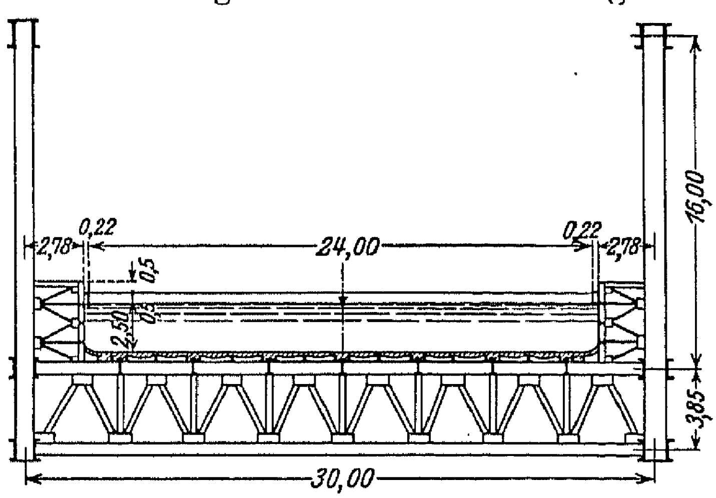


Fig. 550. Cross-section of the canal bridge over the Elbe River. Designed by L. Eilers of Hanover

The tow paths are provided at the sides of the top edge of the trough; they are 2.7 m. (8.9 ft.) wide at both Minden and Seelze. The author wrote a memoir for the same firm concerning the construction of the central line of the Midland Canal which contained a design of a canal bridge over the Elbe. It was found that a bridge of 100 m. (328 ft.) length could readily be constructed. In this case the canal trough was 30 m. (98 ft.) wide. Heavy main trusses at the sides connected by transverse trusses were proposed (Fig. 550). The investigation showed that such bridges might be constructed to lengths of over 100 m. (over 328 ft.) in spite of the heavy load which must be supported.

e. Canal Tunnels

The construction of two-ship canal tunnels for canals of the first order is impracticable because of the excessive cost and uncertainty involved. The discussion concerning one-ship and two-ship canals in connection with bridges applies also to tunnels. The most suitable shape for a tunnel is practically always more or less of a circle or ellipse. If it is desired to provide a minimum breadth for a two-ship tunnel serving ships of 10.5 m. (34.4 ft.) breadth, at least 25 m. (82 ft.) clear breadth should be provided; however, considerable danger of collision s brought about by narrowing a two-ship canal tunnel to this amount. The tunnel itself will have to be 30 m. (98 ft.) wide if a tow path of 2.5 m. (8.2 ft.) is to be provided on each side. The height of the tunnel would then be at least 18 m. (59 ft.). Although tunnels of such dimensions are probably possible at the present day, they are still impracticable. Furthermore, two one-ship tunnels would under all circumstances be cheaper and safer than a single two-ship tunnel. In such tunnels the tow path might be located in the top portion. The amount of excavation required for two one-ship tunnels would then amount to approximately three-fifths that required for one two-ship tunnel and the cost would probably be only about one-half that for the two-ship tunnel. Fig. 551 indicates a one-ship tunnel having the tow paths on both sides; Fig. 552, one in which the tow path is above. The towing locomotive may be arranged to go through the tunnel or may be replaced by crabs operated in the top portion of the tunnel.

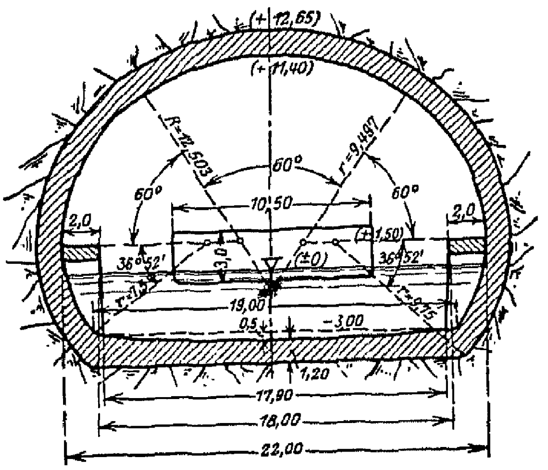


Fig. 551. Tunnel provided with arrangement for towing from the side

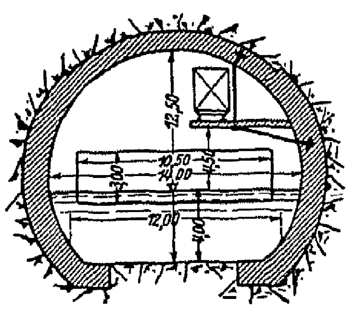


Fig. 552. Tunnel provided with arrangement for towing from above

Figs. 551 and 552. Canal tunnels

E. EXAMPLES OF IMPORTANT WATERWAYS

a. Inland Canals and Canalized Rivers

1. General

This description of important inland waterways is limited essentially to those in Germany. The relation of these waterways to those of

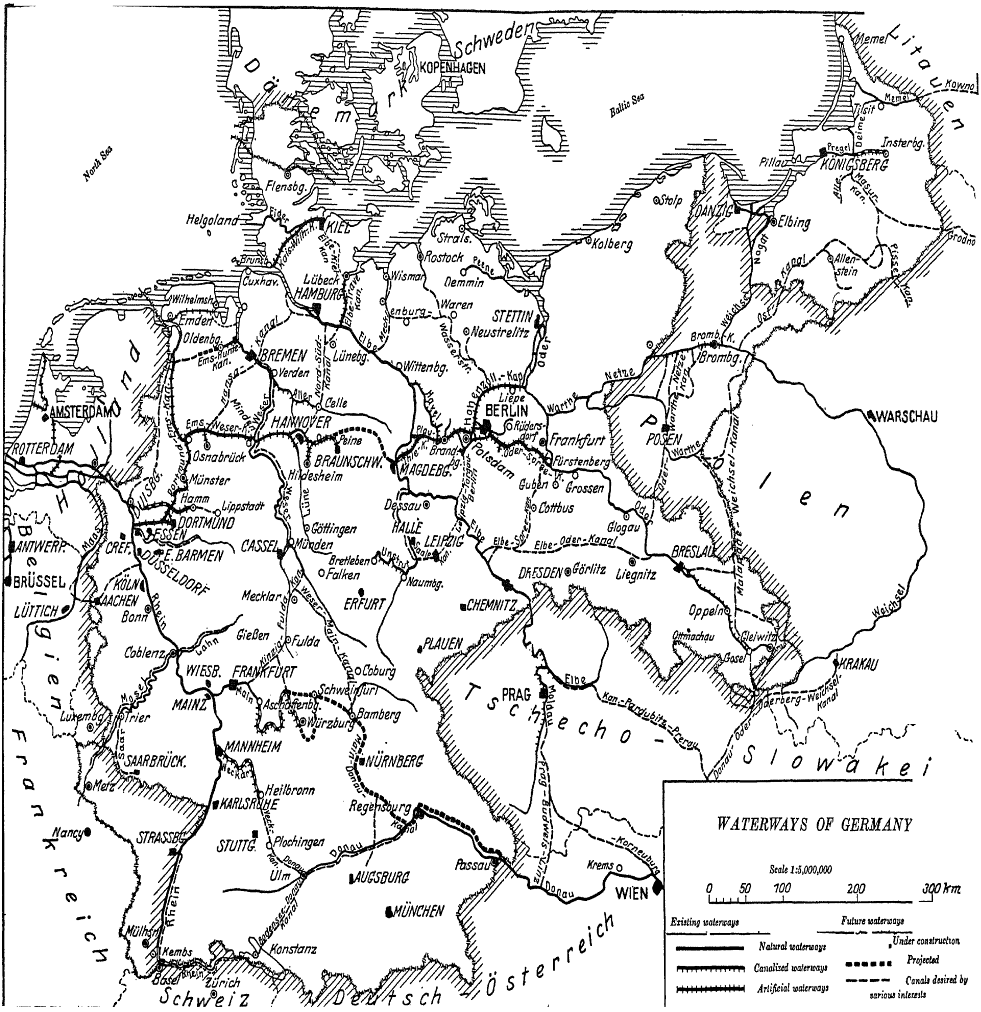


Fig. 553. Waterway net of Germany

neighboring countries is discussed very briefly in a few instances. The German waterway net (Fig. 553) is still in the process of development and presumably will develop at a more rapid rate as soon as Germany recovers more fully from the World War.

There are at present three separate river and canal districts; these include the western district from the Rhine to the Weser with the intermediate Dortmund-Ems Canal, the eastern district from the Elbe to the Vistula, and finally the southern district which consists of the Danube with its navigable tributaries. One of the greatest omissions of the past has been the failure to join the western and eastern districts. One of the canals of the first order is the Dortmund-Ems Canal which ties the Ruhr district to the coastal city of Emden. The canal is navigable for ships up to 800 tons capacity. It drops from Dortmund to Emden with many locks of relatively small lift. Its further development will consist of replacing lock flights by single lifts. Next in importance is the Rhine-Hanover Canal, which leads from Ruhrort to Herne, where it connects with the Dortmund-Ems Canal, and is now completed from Bevergern to Hanover; its extension from Hanover to Hildesheim and Peine is under construction (1927) and will be lengthened over Brunswick to Magdeburg and over the Elbe. From here on follows the Brandenburg net of waterways of which the Plau Canal and the lower Havel development connect the Elbe with Berlin. The Berlin-Stettin Canal connects Berlin to the lower Oder and to Silesia by the Oder-Spree Canal. There is a connection with the Vistula by the Bromberg Canal which joins the districts of the Warthe and Netze Rivers with the Brake district.

In addition to these trunk lines there are a large number of smaller canals which at present are without economic significance. The Saale canalization system which is navigable for 400-ton ships deserves mention. The Saale, with a connection to Leipzig, will be developed as the "south wing" of the Midland Canal. Under the present circumstances, because of its relatively small dimensions, it does not possess important economic significance. The same is true of the canalization of the Fulda, Aller, and others of the smaller rivers.

In the southern waterway district, the Main is developed to Aschaffenburg for 1,200-ton ships and the extension to the Danube is under construction. The Neckar is under development from Mannheim over Stuttgart as far as Plochingen, likewise for 1,200-ton ships.

2. The Midland Canal Project

The existing Rhine-Hanover Canal was developed for 600-ton ships (Fig. 554). In order to simplify supplying water to the canal, the height

of the bridges above normal water stage was made 4.5 m. (15 ft.), so that by adjusting the water level .5 m. (2 ft.) several million cubic meters of feed water could be collected. It has been decided to keep the water level raised .5 m. (2 ft.) continually in the future. The effec-

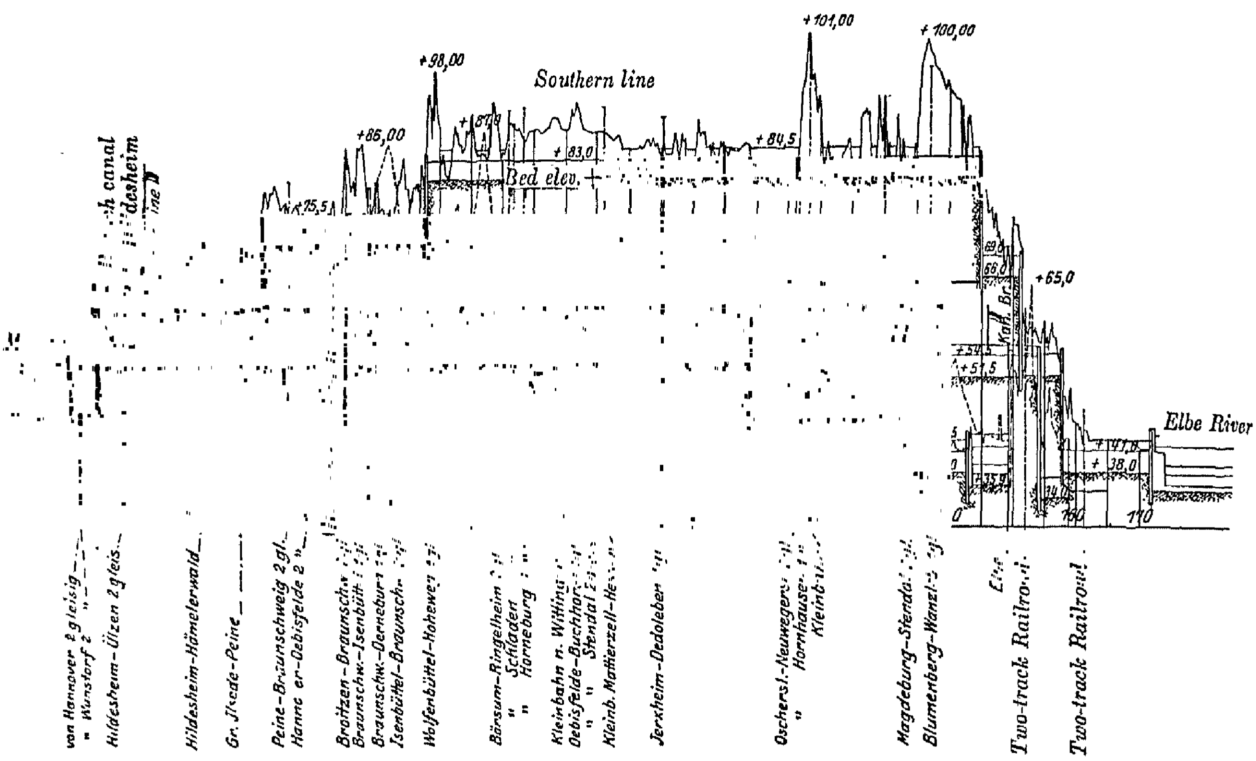


Fig. 554. Designs for the continuation of the Midland Canal beginning at Hanover. Northern line, southern line, central line I and II

tive section is increased thereby to the extent that 1,000-ton barges can navigate on the canal. It thus becomes possible to arrange the existing portion of the Midland Canal (Fig. 554a) for 1,000-ton ships. This occurrence shows the importance of the requirement not to construct arch bridges over a canal.

The prolongation of the Midland Canal follows a course proposed by the author.[1] The author's design was taken over by the government and revised, but the principal features remained unchanged. At Hanover the canal ascends from the Weser level to the summit level [elev. 65 m. (213 ft.)] by means of a lock at Anderten (Hanover) which is constructed as a double barge-train shaft lock with five retention chambers constructed above one another on each side. The canal adjoins the important potassium transportation center, Sehnde, extends to Peine and Brunswick, and then drops to a lower level at a lock in the vicinity

[1] A design concerning the completion of the German Midland Canal by construction of the Central Line, Magdeburg, 1918, printed by E. Baensch, Jr.; memoir regarding the Midland Canal, etc., by the Prussian Ministry of Public Works, February 1, 1920.

of Fallersleben to the Drömling pool at elevation of 56 m. (184 ft.). The canal then traverses the valley of the Aller and Ohre, the Drömling district, thence north of Magdeburg through a 600 m. (1,970 ft.) canal bridge across the Elbe. Here at Hohenwarthe lies a lock for descending to the Ihle Canal. The lock has a lift of 18.6 m. (61 ft.) at an elevation of the Ihle Canal of 37.40 m. (123 ft.). As a result of backwater due to wind, the lock lift may reach a maximum of 20 m. (66 ft.). The Ihle Canal is to be developed eastward past Burg. Shortly before Genthin is reached there is a drop to the Plau Canal which connects the Elbe with the Havel.

A spur canal extends from the main line to Hildesheim; it is approximately 15 km. (9 mi.) long and at elevation +74 m. (243 ft.). A spur at Brunswick is unnecessary since the main line lies near enough to that city. It is planned to have connections by means of descending locks to the Elbe at both sides so that in case of failure of the canal bridge, transportation over the Elbe is still assured.

The spur canal to the Elbe on the left side will be continued to the Magdeburg harbor (Rothensee). There is also a so-called south wing which is an extension of the Elbe waterway to the mouth of the Saale, the development of the Saale over Bernburg, and a spur canal to Leipzig. At Magdeburg the Elbe will be replaced by a by-pass canal on the right shore of the river. The deepest cut in the course is east of Brunswick, where it measures 21 m. (69 ft.). The embankment through the lowlands west of the Elbe will be 6 km. (4 mi.) long and average 15 m. (49 ft.) high. It has not yet been determined whether the east descent should be designed as a lock or as a lift.

The problem of supplying water to the Midland Canal played an important economic rôle in choosing the best course of the three proposed, the north, the south, and the central course. The supply is to be assured by the development of the Bode, Ecker, and Oker impounding reservoirs and also by the large Oder basin. The water required for the canal when approximately fully loaded has already been discussed. The design of the water supply system is explained in the following. The large Rappbode dam, which forms a reservoir of 70 million cu. m. (2,478 million cu. ft.) capacity, is to be connected to the Ecker valley by a pressure tunnel 40 km. (25 mi.) long. Here the tunnel from the Ecker valley (under circumstances also the Oker valley) and from the large Oder reservoir join with the Bode tunnel. The power plants of the individual projects are joined in several steps so that in all, 135 million kilowatt hours can be gained according to the plan of the Elbe River Commission; thereby approximately 4.5 cu. m. (159 cu. ft.) per sec. water is fed into the Midland Canal, this quantity being conducted from the Ecker valley through the Oker valley over Brunswick to the summit stretch. By the construction of mechanical lifts on the east slope, the water will all be at the disposal of the west slope and will also provide excess water to the Weser stretch for feeding the Hansa Canal (to be described later).

The dams provide a maximum fall from the large Oder reservoir to the lowest power plant of over 600 m. (1,970 ft.). The water available will fully provide for seepage, evaporation, and losses in the locks, after mechanical lift on the eastern slope has been constructed. If it should be chosen to use locks on the east slope, it would be necessary to supplement the supply by pumps when the canal is fully loaded. This procedure would not provide any difficulty inasmuch as there is a large amount of energy gained on the project. However, the provinces of Hanover and Saxony have raised objection to diverting water; consequently, for the present, presumably it will be necessary to obtain water by pumping directly from the Weser and Elbe.

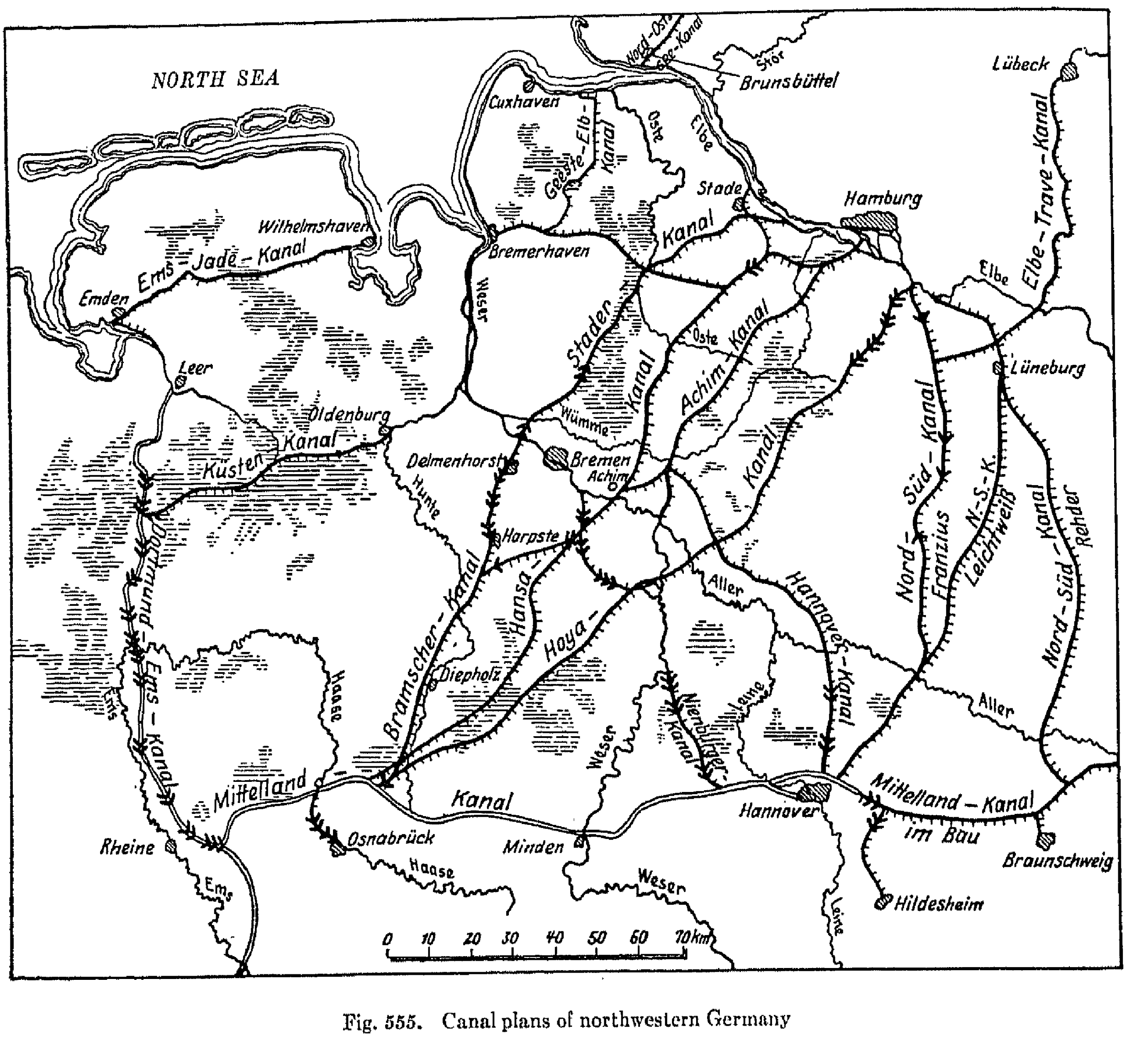
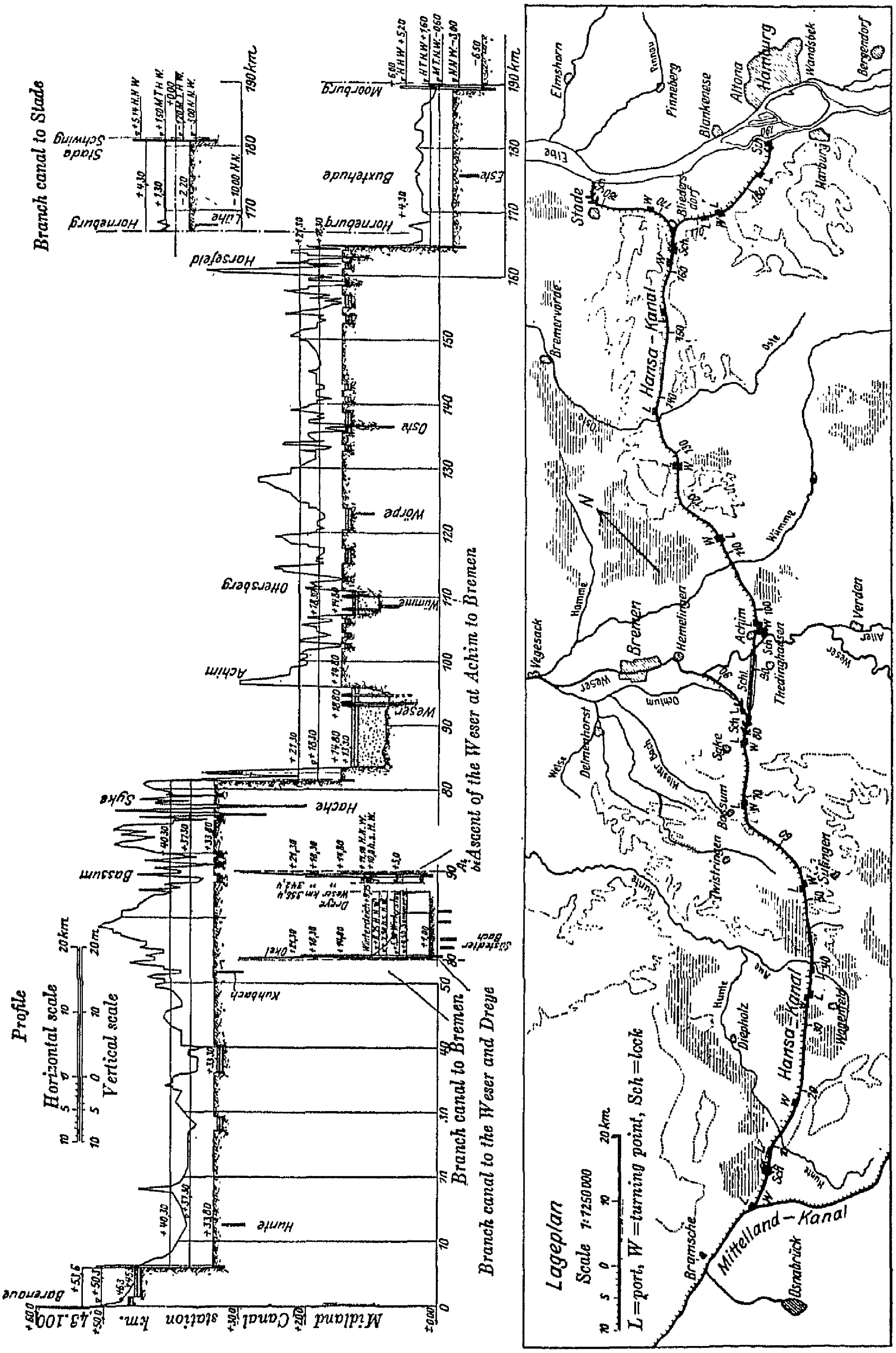


Fig. 555. Canal plans of northwestern Germany

The designs were made on the basis of an estimated volume of transportation of 12 million tons. It is probable, however, that shortly after the completion of the project this figure will be considerably exceeded, making it necessary either to increase the period of navigation per day or to increase the size of the barges. The Midland Canal is so important

for the development of the entire German canal system that it will cause important developments in freightage.



Figs. 556 a and b. Hansa Canal. Plan and profile. Longitudinal scale 1: 250,000. Vertical scale 1:1,250

3. Coastal Canal, Hansa Canal, and the North-South Canal

This group of canals is to effect a connection from Bremen, Hamburg, and Lübeck to the hinterland (Fig. 555). The canals are important sea harbor canals and will be discussed jointly.

a. The Coastal Canal and the Hansa Canal

The Campe-Dörpen Canal, a very old waterway, is being developed into a 600-ton barge canal (Fig. 555) to serve as an outlet for the Oldenburg marshlands. The canal is under construction as far as Campe (1927). Oldenburg is striving against East Frisian interests to have the canal continued to the Dortmund-Ems Canal at Dörpen, and will probably be successful. The canal has exceptional significance in improving the land traversed. It drains the high marshes and makes possible the development of valuable marsh products. Unquestionably it will pay for itself in the improvement to the adjacent land. It will have limited significance as a transportation route for Oldenburg and the lower Weser towns, Bremerhaven, Wesermünde, Brake, etc. The city of Emden fears that it will be affected adversely by the canal. However, the project is not of great importance as a large waterway, it having circuitous course to Hamburg compared to the direct route of the proposed Hansa Canal. The canal will be very cheap since it lies in country which is practically flat.

The Hansa Canal (Figs. 556 a and b) is to be the trunk line from the Ruhr district over the Midland Canal to Bramsche, thence over the existing Hansa Canal. As shown on the map, it forms a practically straight connection, having only a slight buckle at Bevergern. The canal continues from Barenaue (Bramsche) to Hamburg without loss in fall. It crosses the Weser by means of a canal bridge; a descent to the Weser is also provided. The Hansa Canal has three descending locks to the Weser and a total of four to the Elbe. The canal is to be developed for 1,000-ton ships. Its significance rests principally in its use for transporting coal to the sea harbors. It will also serve as the transportation way between Bremen, Hamburg, and Lübeck. Finally, in case of damage to the Midland canal bridge in Minden, or of a break in the canal levee in that vicinity, it makes possible replacing the broken stretch by a detour over the Hansa Canal and Weser River. Next to the Midland Canal, the Hansa Canal is the most important German canal. It encourages further growth of the German sea harbors, the development of which is necessary for the economic life of the hinterland. In this respect the Hansa Canal is of much greater significance than any of the local canals.

TABLE NO. 21

LENGTHS OF VARIOUS CANAL STRETCHES FROM GELSENKIRCHEN

From Gelsenkirchen to	Roadway	On the Midland Canal the canalized Weser or Elbe		Coastal Canal		Hansa Canal over "Bramsche Achim"		North-South Canal Leichtweiss		North-South Canal Franzius	
	km.	Actual length km.	Equivalent operating length km.	Actual length km.	Equivalent operating length km.	Actual length km.	Equ valent operating length km.	Actual length km.	Equivalent operating length km.	Actual length km.	Equivalent operating length km.
Bremen	245	338	410	337	413	264	296	...	...	...	...
Hamburg	345	711	743	435	523	346	378	452	484	423	468
Lübeck	411	723	783	562	676	473	533	485	545	488	556

The lengths of various canal stretches from Gelsenkirchen in the Ruhr district are given in Table 21.[1]

β. The North-South Canal

The North-South Canal (Figs. 557 a and b) was first planned by Oberbaudirektor Rehder, Lübeck, for broadening the hinterland of Lübeck and also for Hamburg, and furthermore, to provide a junction of these two sea harbors with the older northern line of the Midland Canal. The work of Rehder, *Der Nord-Süd Kanal u. das mitteldeutsche Kanalnetz*, is one of the largest and most important works with regard to a canal network that was ever published. The canal was to extend from the Elbe to Lauenburgs by way of Lüneburg, Ulzen, and Giffhorn. Among other things, the project was objected to largely because of coal transportation from the Ruhr district to Hamburg. The Rehder course was revised by the author with a line from Hanover over Celle to Hamburg which, just as the Rehder line, provided for a branch to Lüneburg for the purpose of a connection with Lübeck.

The Franzius course of the North-South Canal crosses the water divide between the Weser and Elbe Rivers in a summit stretch at elevation +80 m., while the Rehder route descended from the pool of the North line elevation +56.6 (later that of the Central line +65) to the Elbe without loss in head. A comparison of the two designs shows very clearly that the designer should not be influenced by a loss in head of water. The economic solution is of greater importance than the technical aspect. The North-South Canal, Hanover-Celle-Hamburg, as compared to the solution by Rehder, gives a saving of over 73 operating km. (over 45 mi.); that is, inclusive of time loss through locks. It is more expensive than the solution by Rehder, but more economical. The summit level cannot be fed naturally in the Franzius solution. Even this factor is negligible as compared to the tremendous advantage of the shortening introduced. It would be necessary here to use inclined planes or mechanical lifts instead of locks. An intermediate proposal was made by Professor Leichtweiss of Brunswick, which joins the courses of Franzius and Rehder going from the first to the second in a pool at elevation +60. But this line also involves a detour for Hanover amounting to 16 operating km. (10 mi.). However, the Leichtweiss course would probably be less expensive than the Franzius course.

In the further development of the German canal system it will be necessary, by means of precise economic investigations, to determine the advisability of the indirect route to Hamburg. A comparative estimate of the two schemes is important since the lock lifts of 20 m. (66 ft.) each of line II compared to the 17 km. (11 mi.) greater length of line III will have great influence upon the choice. It is possible that the course proposed by Leichtweiss is the most economical, but this still remains to be proven. At present the early construction of a North-South Canal is not under consideration.

[1] Teubert, *W. Reederei und Hafen*, 1924, Vol. 13; Leichtweiss, *Verkehrstechnik*, 1924, Vol. 31.

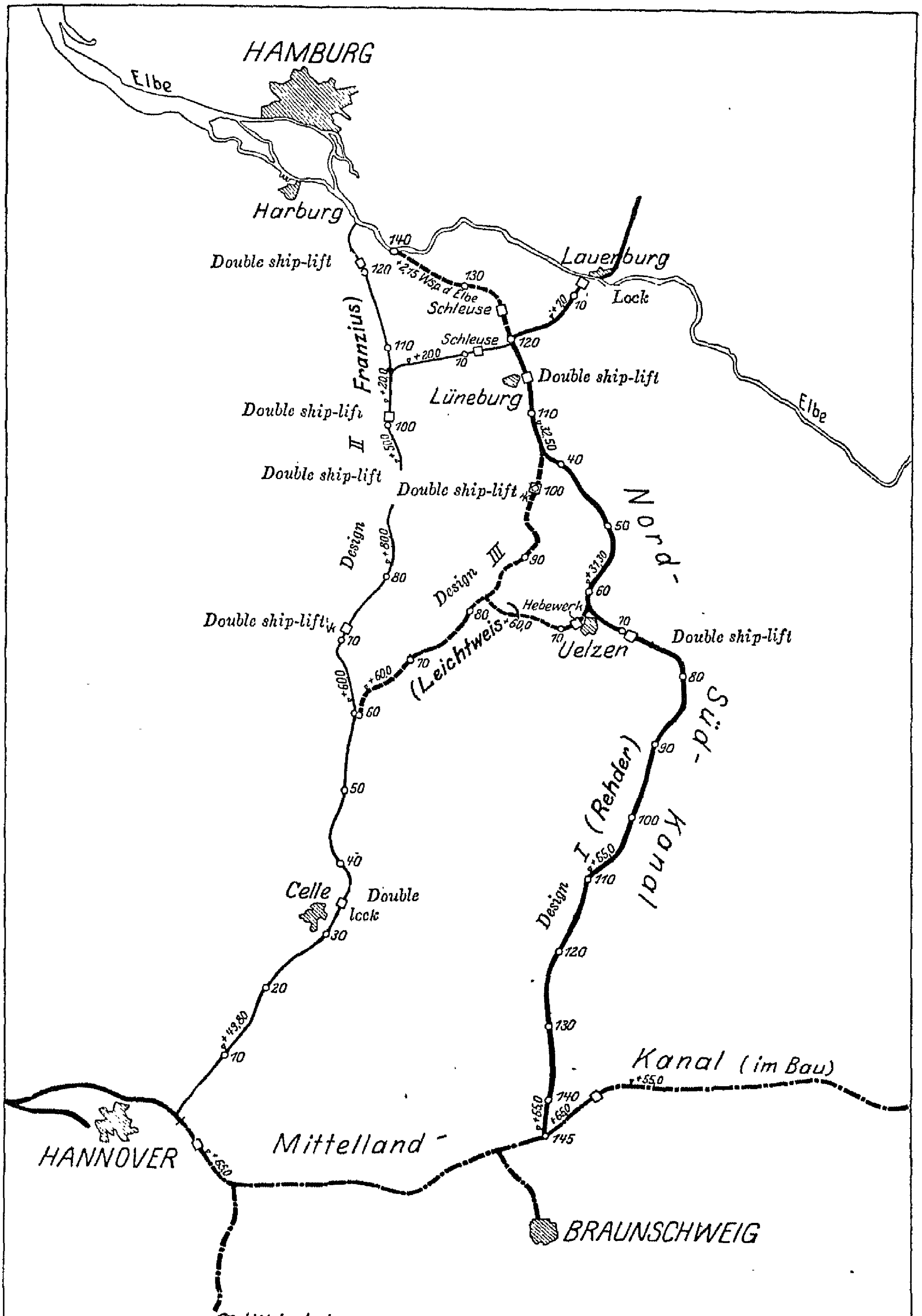


Fig. 557a. Plan of "North-South" Canal

Schleuse = Lock. *Nord-Süd Kanal* = North-South Canal
Mittelland Kanal (in Bau) = Midland Canal (under construction.)

4. The Weser Canalization and the Weser-Main Canal

The lower part of the Weser canalization affects the Hansa Canal situation. It is possible to replace the Hansa Canal to some degree by canalization of the Weser from Bremen to Minden and by using the Midland Canal from Minden to Bramsche. Large quantities of electrical energy would be gained as a result of the canalization. It was also believed for some time that the largest part of the construction cost would be carried by the energy gained. Today (middle of 1927) the current generated by lignite is available at such a low rate that the Weser canalization is no longer justified on an energy economic basis. Hence the tolls which would have to be collected on the canalized Weser would be about the same as those on the Hansa Canal and the possibility of a cheap connection of Bremen, Minden, and Bramsche would disappear. Because of the many bends in the Weser, the course from Bremen to Minden over this route, even after canalization, would be longer than over Bramsche-Minden (Hansa Canal and Midland Canal). Consequently, Bremen would not gain any particular advantage for its upper Weser influence area in the Weser canalization because after the construction of the Hansa Canal ships would no longer use the Weser.

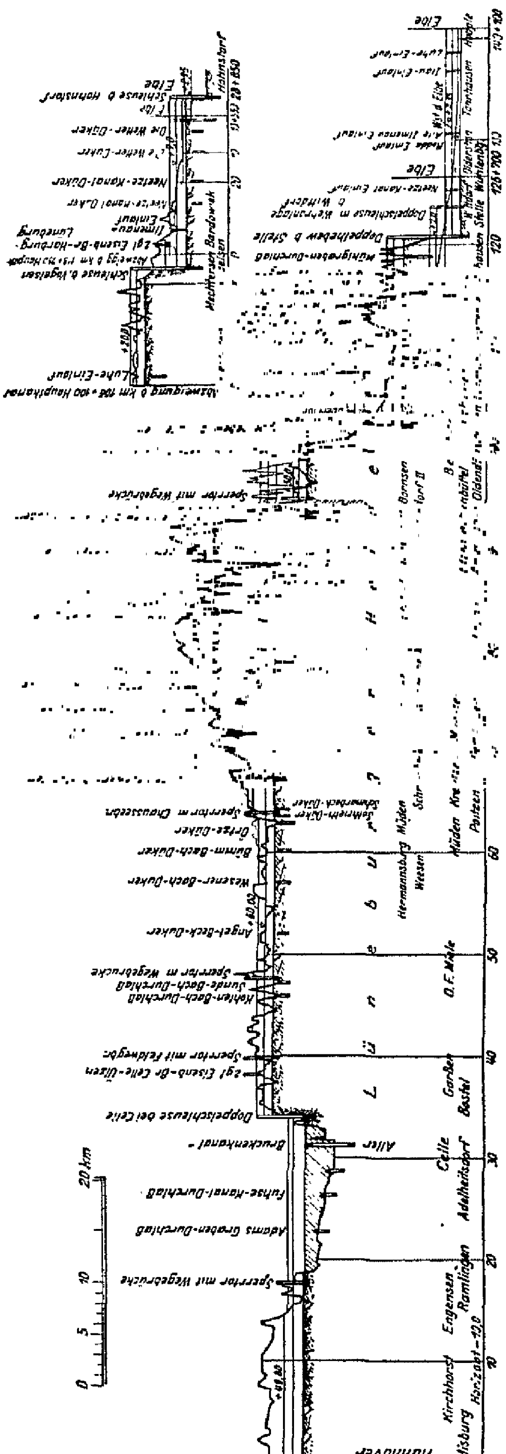


Fig. 557b. Profile of "North-South" Canal. Horizontal scale 1:1,000,000. Vertical scale 1:3,000

The canalization of the Weser from Minden upstream to Hanover-Muenden, and the reconstruction of the old Fulda canalization to Kassel, possibly also canalization of the Werra to Berka in the near-by potassium district, is a problem of great economic significance. The same cannot be said with regard to construction of a Werra-Main or Fulda-Main

Canal. Estimates of the State Planning Commission in Eisenach, according to a report by Inecken,[1] indicate that the most favorable of these courses would result in a deficit of 14 million gold marks annually. The present amount of transportation would have to increase 5.2-fold if the invested capital is to pay for itself. It is possible that, after the development of the large German canal net, the Werra-Main or Fulda-Main Canal will become an indispensable connecting link between the north and south.

The Werra course branches off from the uppermost Weser pool at elevation 117 m. (384 ft.), rises to elevation 356 m. (1,168 ft.), hence a rise of 239 m. (784 ft.), and then falls toward Bamberg to elevation 242.8 m. (796.6 ft.) to the Danube-Main Canal, making a total drop of 113 m. (371 ft.). The total height surmounted is 352 m. (1,155 ft.). There are 39 locks and 1 inclined plane. The descent requires 9 lifts, of which 3 are mechanical lifts and 6 are locks. The layout is designed for 1,000-ton ships. It is not practicable to feed the peak stretch from impounding reservoirs. Water must be pumped to supply a maximum demand of .3 cu. m. (10.6 cu. ft.) per sec., an excess of 15 liters per sec. per km. (.8 cu. ft. per sec. per mile). The course first follows the Werra valley, reaches the summit pool at Ritschenhausen, and descends through the Kreck, Rodach, and its valleys to the Main, the total length from Hanover-Muenden to Bamberg being 285 km. (177 mi.). The towns adjoining the canal are Witzenhausen, Soden, Eschwege, Ritschenhausen, Romhild, Heldberg, Sesslach, and Hallstadt.

The Fulda-Main route likewise ascends from the last Weser pool, elevation 117, to the summit pool, elevation 350 m. (1,148 ft.), and descends toward Hanau to the Main, elevation 98.91 m. (325 ft.). The total rise in the course is 233 m. (764 ft.), the drop, 251 m. (824 ft.), or a total of 484 m. (1,588 ft.) within a combined length between Hanover-Muenden and Hanau of 232 km. (144 mi.). A number of counter-balanced mechanical lifts providing a lift of 50 m. (164 ft.) are planned for this canal. The practicability of this type of lift has not yet been verified.

5. Development of the Rhine

Large portions of the Rhine and its tributaries are still unimproved. The Ruhr, Lippe, and Lahr have already been canalized. These streams provided for ships of only 165 to 190 tons capacity. Important work in the development of the Rhine district has already been begun on the Rhine-Herne Canal, the Lippe side canal, and in the canalization of the Ruhr as far as Mühlheim for 1,700-ton ships, and for the canalization of the Main and Neckar for 1,200-ton ships. The Rhine-Herne Canal is also navigable for 1,700-ton ships. Canalization of the Ruhr is planned as far as Hagen. Designs have been prepared for canalizing the Moselle and the Saar Rivers for 1,500-ton ships. The most important connection, the Main canalization in conjunction with the Main-Danube Canal, will be discussed later. In order to fully evaluate a connection of the

[1] *Wasserstrassen Jahrbuch,* 1924, Richard Pflaum, Publisher, Munich.

Danube-Main Canal and of the Neckar to the Rhine waterway, the water depth of the Rhine at mean low water which amounts to 3 m. (9.8 ft.) between the mouth and Cologne, and 2.5 m. (8.2 ft.) to St. Gaar, must be deepened to 2.5 m. (8.2 ft.) as far as Mannheim. At present the depth on this stretch, up to Strassburg, is 2 m. (6.6 ft.). It is expected to attain a depth of 2.5 m. (8.2 ft.) in the Rhine to Strass-

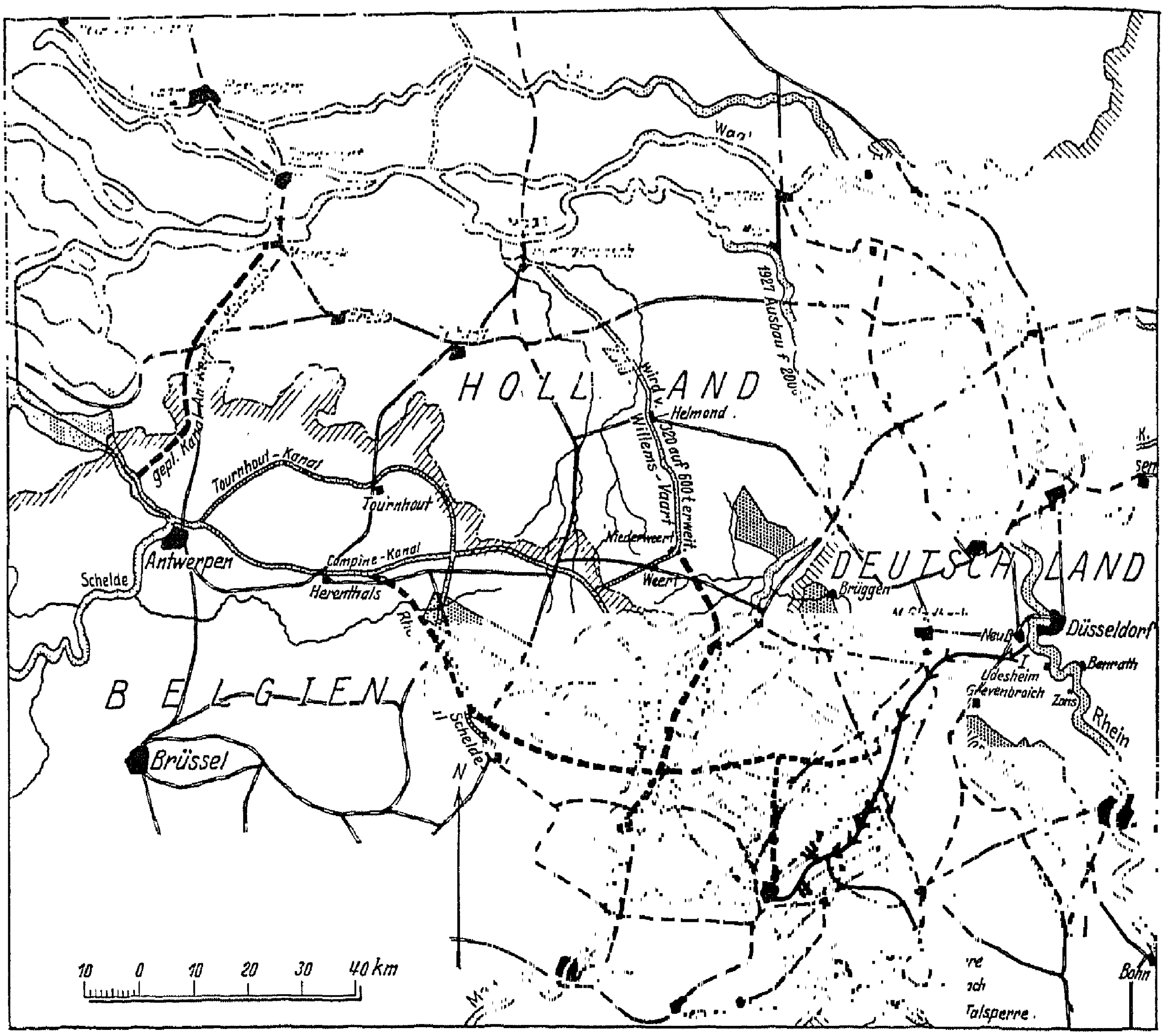


Fig. 558. Rhine-Maas-Schelde Canal

burg and, if necessary, in the upper stretches by canalization. The most important problem in this connection is the development of the Bingen Gap (*Binger Loch*) into a two-ship stretch of 2.5 m. (8.2 ft.) depth. From Strassburg to Lake Constance either canalization or the construction of a side canal is required for obtaining the desired depth; because of the large dimensions of the Rhine, canalization is preferable. France is striving to make the Rhine valuable also for its country by means of long side canals. A design competition for the canalization of

the Rhine between Basel and Lake Constance took place in 1919.[1] Another important possibility concerns developing the Rhine between its mouth and Coblenz to a depth of 3.5 m. (11.5 ft.) for a bed breadth of 150 m. (492 ft.).

6. The Rhine-Maas-Schelde Canal and the Aachen-Rhine Canal

The Versailles Treaty requires Germany to construct a German connection to the Rhine-Schelde Canal planned by Belgium in case Belgium demands such within twenty-five years after signing the Treaty. A large number of designs for such a canal have been made, some of them by German and others by Belgian interests (Fig. 558). The designs may be divided according to whether they include passage through Krefeld, Münich-Gladbach, or Aachen; and whether they are to be through canals or feeder canals. Recent investigations made jointly by the author, Professor Proetel of Aachen, and Dr. Werner Teubert of Berlin, indicate that economically the best solution would be to construct a canal at a mean elevation of about 86 m. (282 ft.) through the vicinity of Aachen and connected with Aachen, Eschweiler, and Duren by spur canals. In this layout the Maas would be crossed by a canal bridge. Through traffic would have to follow a roundabout course, but the important coal district of Aachen, Eschweiler, Geilenkirchen, Dutch Limburg, and Campine would be connected. The coal district of Rheydt could also be connected to the system. The canalized Maas of Holland and Belgium would also be provided with a waterway connection at mean elevation toward the east and west so that goods from the Rhine as well as Antwerp could be transported over the Maas to Belgium. The study indicated that through traffic would not be sufficient to pay interest on the cost of construction. Transportation on the Rhine is so cheap that through traffic on the canal could be charged a toll of less than one-fifth of that raised on the Midland Canal. The Rhine-Schelde Canal is in a class by itself because it competes with the cheap Rhine course rather than with the railroad. In spite of the great shortening of the course, the route over the Rhine would be preferable in many instances. The plan of constructing a Rhine-Schelde Canal has been laid aside for the present. There is now some consideration of constructing a canal between the Aachen industrial district and the Rhine. This plan, according to a proposed design by Professor Proetel, offers so much of interest that it is worthy of a short discussion. The canal is 62 km. (39 mi.) long and would require 180 million gold marks inclusive of pump layouts for feeding the channel. According to statements of coal mining companies and accurate figures on transportation

[1] de Thierry, *Bauing.*, 1921.

costs, at least seven million tons transportation is to be expected annually. In order to pay interest on the investment for the canal, toll rates of about two pfennigs per ton-km. would have to be charged; this is four times the rate formerly charged on the Midland Canal for Class V goods. Such a canal would make it possible to transport goods directly from the Aachen district to Mannheim and other Rhine cities without reloading.

The following comparative figures are given for freight on one ton of coal:

Entirely by railroad from Alsdorf to Mannheim. . . . . 10.90 marks
By train to Cologne, then by ship to Mannheim, and there again
reloading to the train. . . . . 7.30 marks
Entirely by ship to Mannheim with reloading costs to the railroad
at Mannheim. . . . . 5.20 marks

In this comparison, the last figure contains a toll of two pfennig per ton-kw. on the Aachen-Rhine Canal. The analysis is of particular interest because it shows that the connecting canal, even with high toll rates, makes possible a great reduction in transportation rates because of the low freightage expenses on the Rhine. The saving is so large that coal from the Aachen district could be

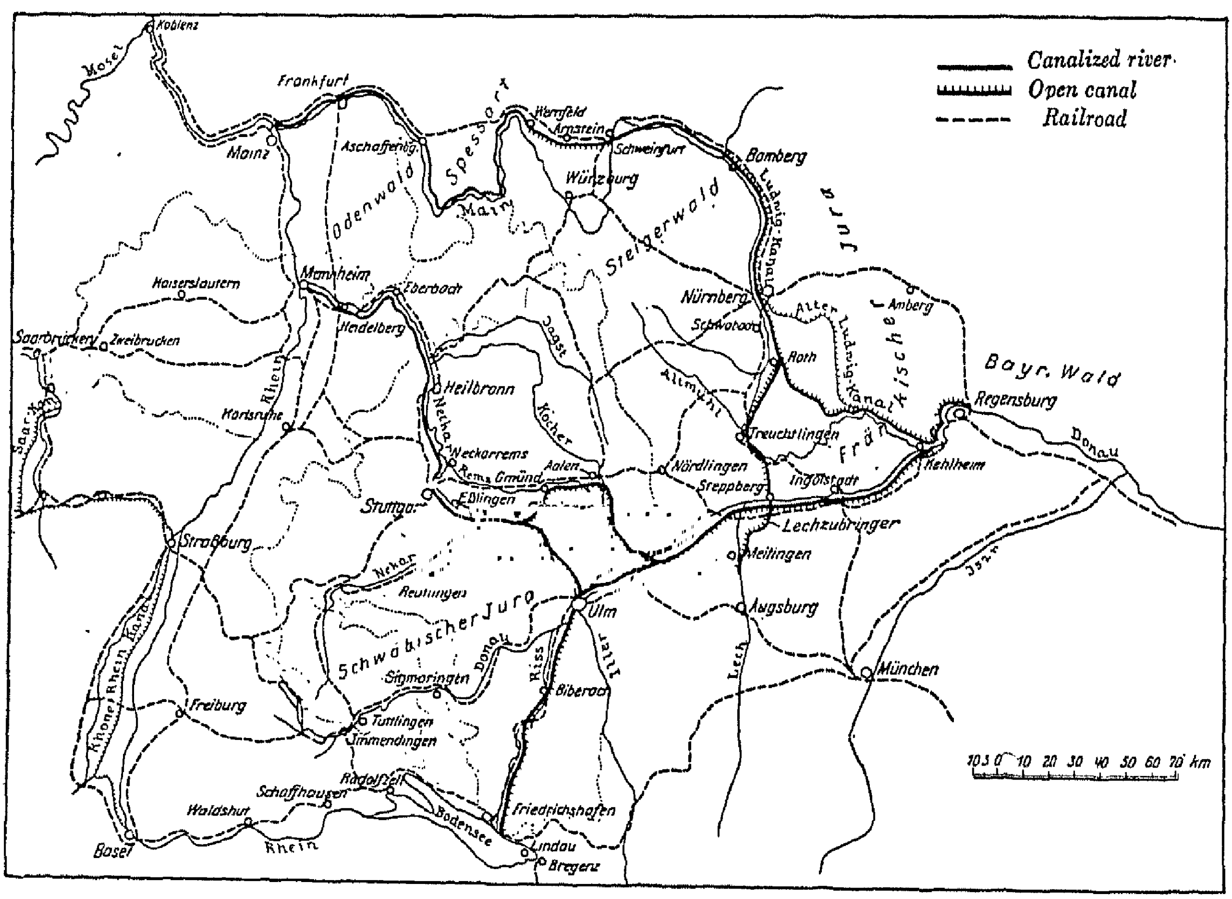


Fig. 559. Canal plans for Southern Germany. Rhine-Neckar-Danube-Lake Constance Canal and Rhine-Main-Danube Canal

7. Canal Plans for Southern Germany

a. Danube-Main Canal

Fig. 559 shows a plan view of the canal system in southern Germany. The forerunners of the Rhine-Main-Danube Canal include the Fossa Carolina north of Treuchtlingen, which was begun in 1793 but not completed, and the Ludwig Canal which was constructed (1836–46) for 120-ton ships. Because of its small dimensions, this canal has succumbed to railroad competition. A large number of plans have been proposed. Of these the Beilngries route over Frankfurt, Aschaffenburg, Wertheim, Gmünden, Arnstein, Bamberg, Nürnburg, Beilngries, Kehlheim has been adopted. This course is indicated in the plan view of Fig. 560. It is compared with the other routes in the summary which follows.

The course now under construction (1927) traverses the large Main loop over Wertheim to Gmünden, then follows the Wern valley eastward, crosses the Main below Schweinfurt, and makes use of the Main River again from Hassfurt to Bamberg. Würzburg, an important city in this region, is to be connected by canalization of the Main for a distance of 35 km. (22 mi.). In addition, a canal in the valleys of the Regnitz, the Rednitz, and Altmühl Rivers is planned over Forchheim, Erlangen, Nürnburg, Roth to a 14 km. (9 mi.) summit pool at Hilt-

TABLE NO. 22

Canal Plans for Southern Germany

Designation of Course	Beilngries Course		Amberg Course[1]	Neuberg Course[2]
		Main Canalization Wernfeld-Würzburg		
Length of waterway	607	35	530	745
Elevation of peak stretch	405	...	425	409.5
Number of locks	49	3	48	54
Number of tunnels	....	....	6	3
Operation length with additions for locks and tunnels	810	52	724	994
Construction costs (without interest during construction) in million marks without providing water for entire stretch	467.9	17.4	651.2	577.4
(*a*) For providing water	47.6	....	11.9	12.5
(*b*) Inclusive of providing water for entire stretch	515.5	17.4	663.1	589.9
(*c*) For km. length of construction	0.85	0.5	1.25	0.79
Operation and maintenance in million marks	3.4	0.1	8.0	3.9

[1] [illegible] boundary.
[2] [illegible] boundary.

poltstein, elevation 405, and then further over Beilngries to Kehlheim on the Danube. The Danube will be canalized from Kehlheim to below

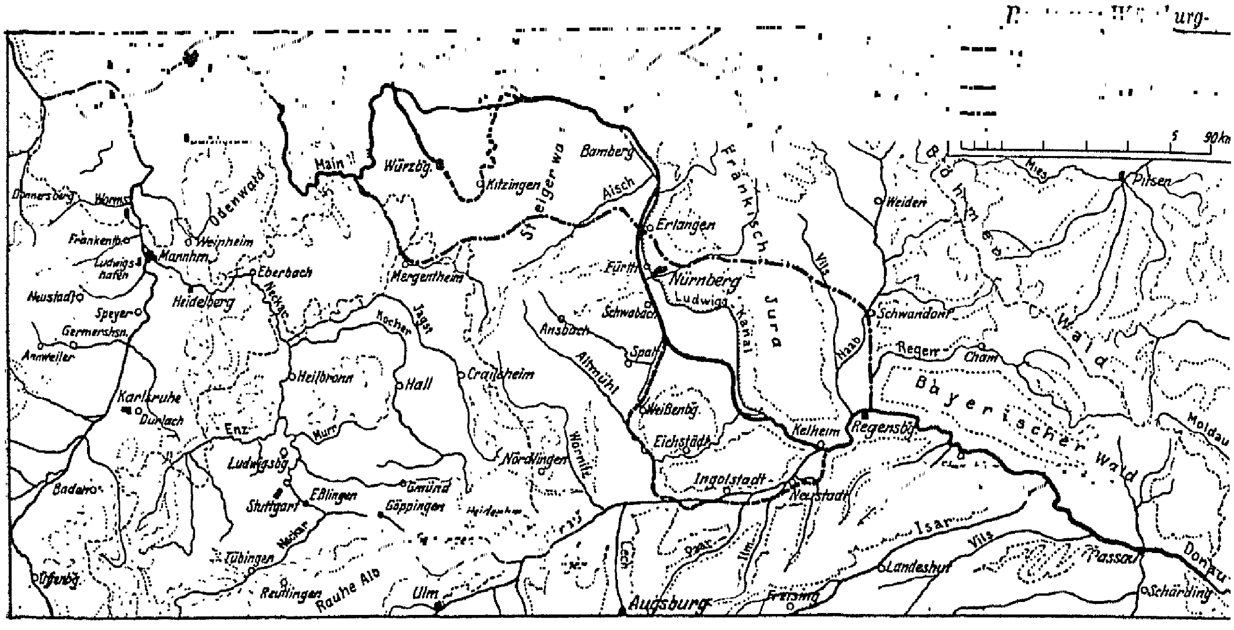


Fig. 560a. Plan of the Danube-Main Canal

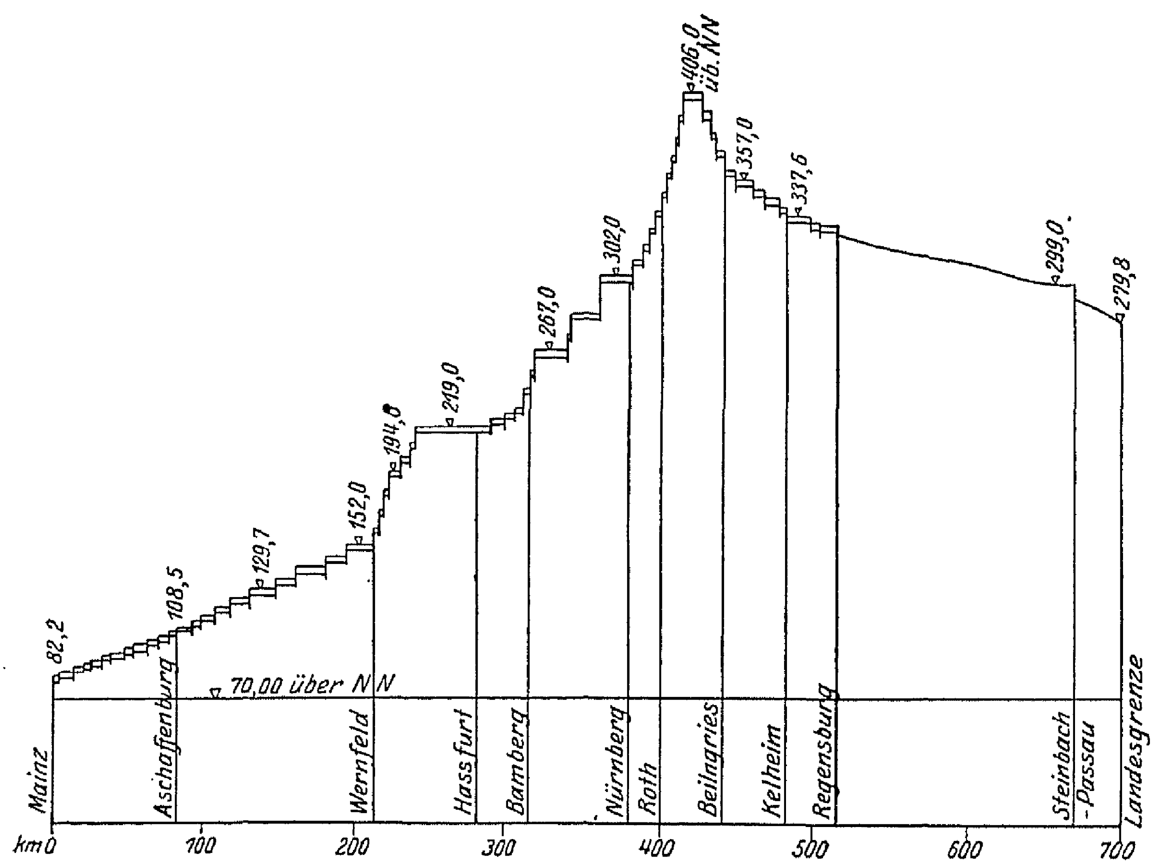


Fig. 560b. Profile of the Danube-Main Canal
Horizontal scale 1: 6,500,000 Vertical scale 1: 5,800

Fig. 560. Trunkline waterway between Aschaffenburg and Passau.

Regensburg or will be replaced for a short stretch by a side canal; the navigable depth planned is 2.5 m. (8.2 ft.). Beginning at the mouth of the Regen (Regensburg), the Danube will be used as a regulated

river having 2 m. (6.6 ft.) water depth at the lowest navigable water stage and a breadth of 80 to 100 m. (262.4 to 328 ft.). A total of about 160 km. (99 mi.) will be improved by the regulation. The breadth on the gravel stretch between the Isar mouth and the German boundary (Passau) increases from 100 to 210 m. (328 to 688.8 ft.). The *Danube-Kachlet*[1] from Pleinting to Passau will be made navigable by the construction of a lock of 9 m. (30 ft.) lift at Steinback above Passau. A longitudinal section is shown in Fig. 560 b. Cross-sections through the pool and shoal stretches are shown in Figs. 561 a and b. A channel breadth of 90 m. (295.2 ft.) is estimated for a minimum depth of 2 m. (6.6 ft.).

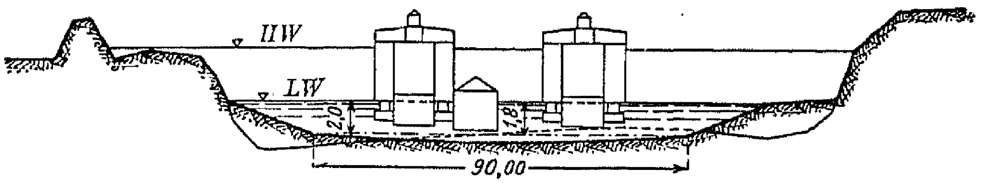


Fig. 561a. Cross-section of the Danube at a point where sills were used

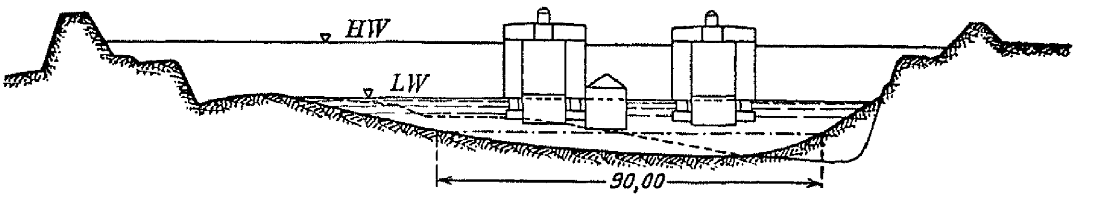


Fig. 561b. Pool cross-section of the Danube

Fig. 561. Cross-sections through the low-water regulation works of the Danube

According to a plan by Hallinger of Munich, the water supply of the canal is arranged in conjunction with a large-scale power development. This project proposes diverting the Lech River above its mouth at Thierhaupten and conducting it over the Danube through a trough bridge at Steppberg. In all, 33 power steps are planned in the entire waterway, 15 in the canalized Main, 11 in the canal course, 4 along the Altmühl, and 3 on the Danube. The useful fall from the summit pool Aschaffenburg amounts to 300 m. (984 ft.); in the Altmühl 18 m. (59 ft.). From Aschaffenburg to Kehlheim there will be a total of 250,000 HP capacity and 1.5 billion kilowatt hours developed annually. In addition to these plans, there is one for the development of the Ulm to Kehlheim principally through side canals with 150,000 HP and 1 billion kilowatt hours annually. Together there can be generated 400,000 HP with 2.5 billion kilowatt hours annually. The significance of this transportation course extends far beyond the pure transportation question. The plan is a civic improvement of the first order in energy generation as well as in navigation. According to estimates of the year 1913, a total annual transportation of 3.3 million tons is to be expected, but it is estimated that by 1955 the annual tonnage will increase to 10 million.

β. The Danube-Neckar-Lake Constance Waterway

This waterway is projected as a future development of which a 200 km. (124 mi.) stretch between Mannheim and Plochingen is under con-

[1] River rapids with rock crags.

struction. The Neckar will be deepened here to 2.5 m. (8.2 ft.) for 1,200-ton ships. The total drop of 160 m. (525 ft.) will be utilized for energy generation in 28 steps; approximately 5 billion kilowatt hours are expected to be gained annually.

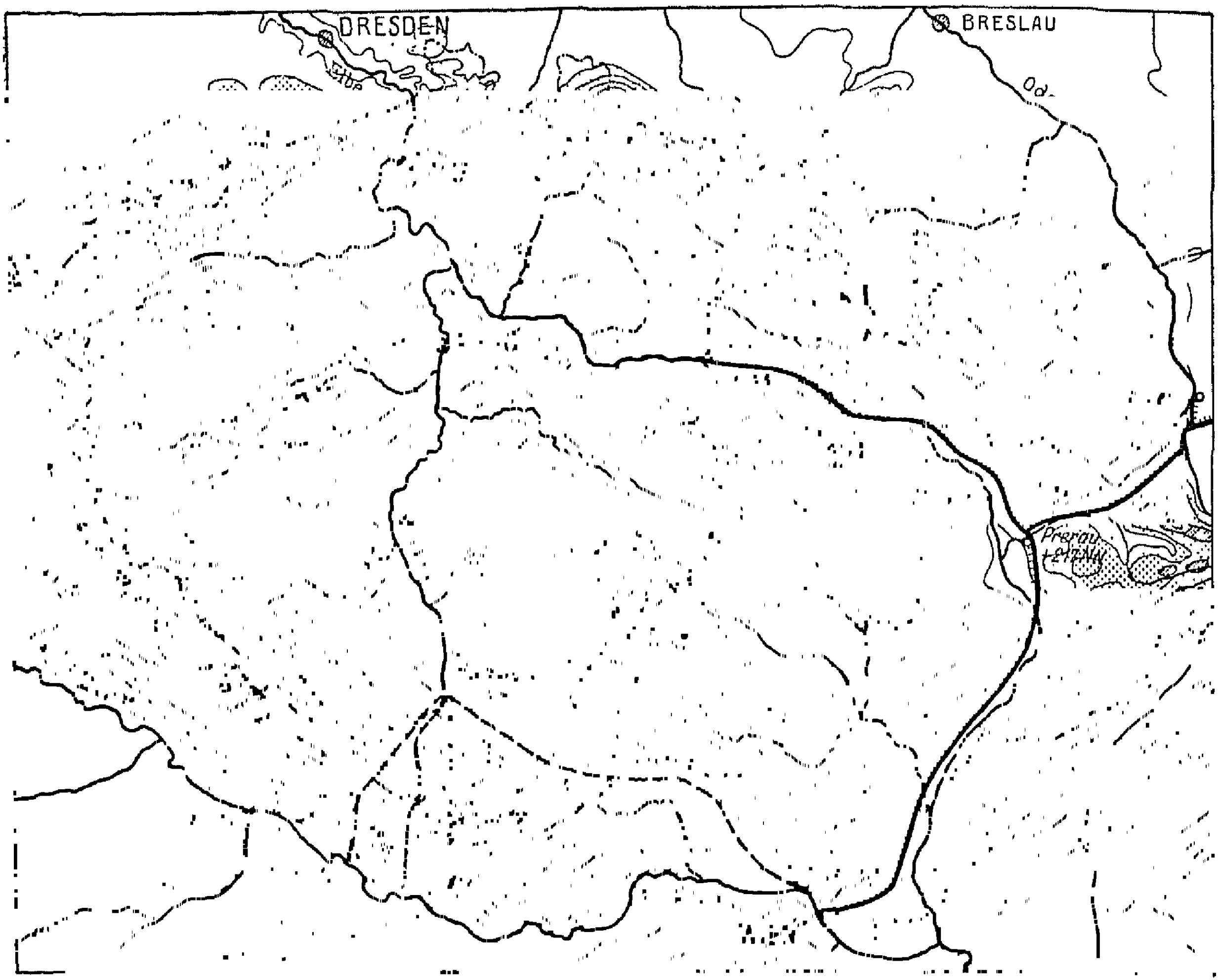


Fig. 562. Elbe-Danube Canal plans

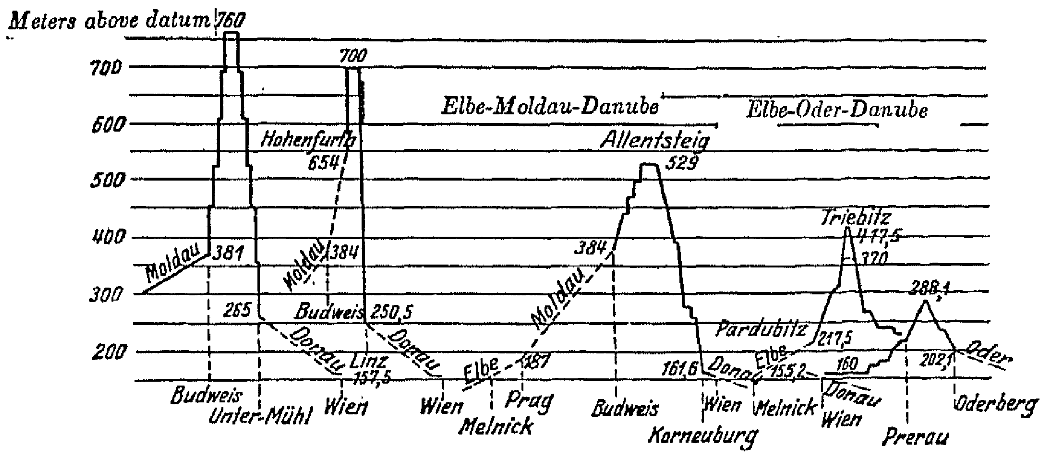
It is intended to continue the canal from Plochingen to the Danube at Ulm. The valleys of the Rems and Brenz streams can be used with a summit pool at elevation 496 m. (1,626 ft.). Other solutions are also practicable. Since the Danube is to be canalized from Ulm to Kehlheim, a valuable connection to the east is thereby provided. The continuation to Lake Constance can be made upstream through the Riess valley and downstream in the Schussen valley toward Friedrichshafen over a summit level of about 30 km. (19 mi.) length. The Danube at Ulm lies at elevation 464 m. (1,522 ft.), the summit pool at 546 m. (1,791 ft.) and Lake Constance at 395 m. (1,296 ft.).

8. Connection of the Elbe and the Oder with the Danube

This waterway is of significance for Hamburg, but the construction is possibly far off because of political entanglements. The course is indicated in the plan in Figs. 563 and 564.[1] A canal is planned to extend

[1] Bubendey; *Die Elbschiffahrt u. ihre Fortsetzung zur Donau*, 1916, Herold's Book Company, Hamburg.

from the canalized Elbe to Pardubitz, reaching the summit level, 417 m. (1,368 ft.), at Trebitz and then lowering in the March valley over Prerau



Donau = Danube

Fig. 563. Elbe-Oder-Danube Canal plans. Profile

to elevation 217 m. (712 ft.). Vienna is to be reached at Floresdorf. Among other reasons, the course is of value because it forms a connection of the Oder from Oderberg to Prerau on the Elbe over a short distance of approximately 80 km. (50 in.). In this manner a star-shaped waterway would be created between the three large river districts.

A longitudinal section of this waterway is shown in Fig. 564. The summit pool of the Oder-Danube connection lies at elevation 288.1 m. (944.9 ft.). Other plans have been formulated in Prague whereby either the Bohemian forest with a pool at elevation 700 m. (2,297 ft.) or the saddle between the Bohemian forest and the Bohemian-Moravian heights are to be crossed at elevation 529 m. (1,736 ft.). These plans are uneconomical as compared to the Prerau line.

9. Further German Canal Plans

In addition to those already mentioned, a group of other German canal plans are of interest. The space of this treatise does not allow discussing in detail. One which should be particularly mentioned is the great German canal route which was designed by Rehder of Lübeck as his last great life work. A longitudinal section of the canal is given in Fig. 564. It is intended to extend from Frankfurt over Schluchtern, Hersfeld, Eisenach, Weissensee, Sangerhausen, Halle (Leipzig), Torgau, Spremberg, Liegnitz, to Breslau. Thus a central route parallel to the Midland Canal and the Main-Danube waterway would be formed. The development of such a canal still lies in the distant future but it indicates a good solution of transportation problems in Germany. Another to be mentioned is the proposed link between the Elbe and the Oder over Kottbus and Guben to Fürstenberg and Frankfurt on the Oder, which was proposed by Platzmann to form a short passage from

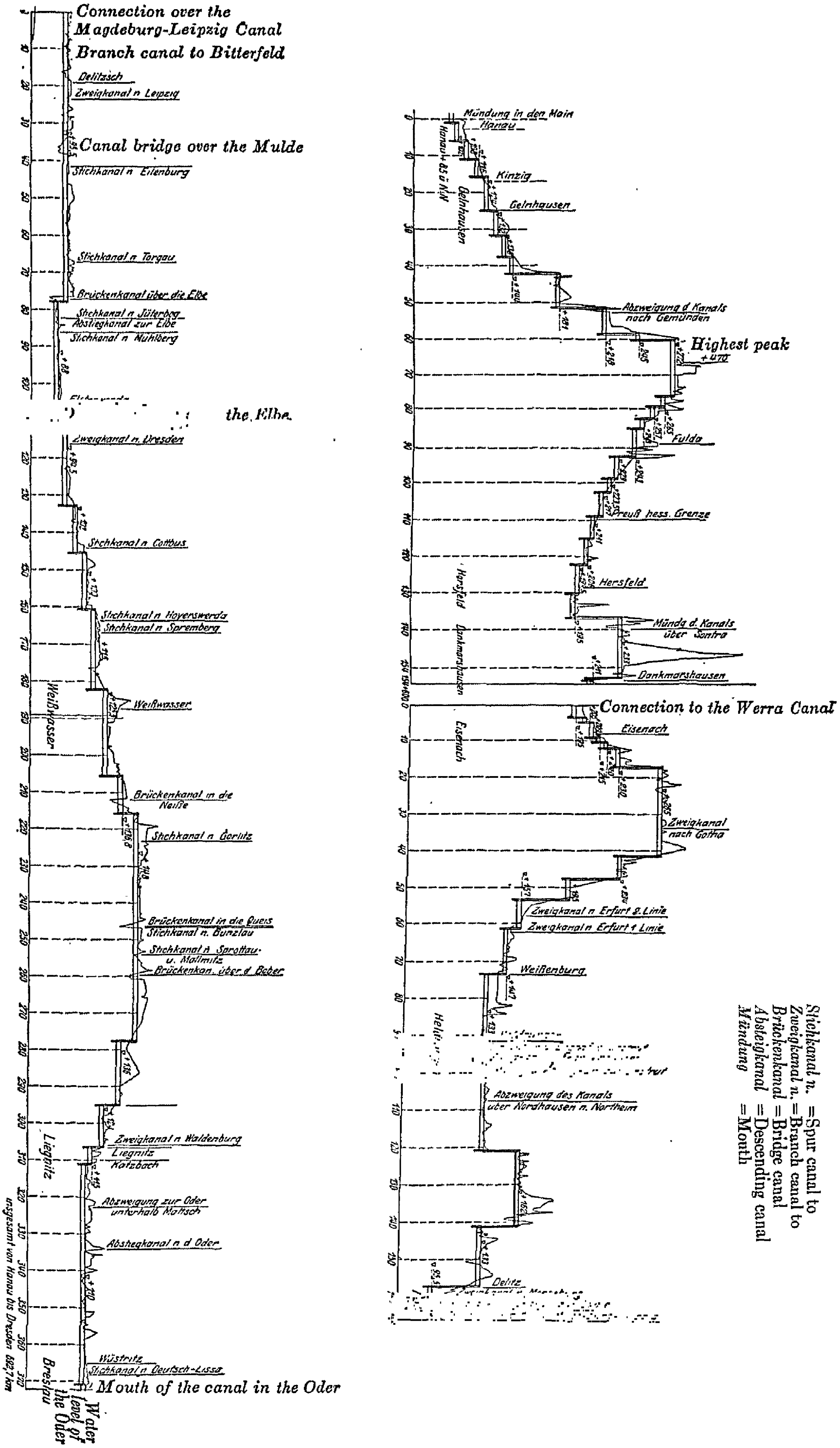


Fig. 564. Central German canal route proposed by Rehder. Profile Horizontal scale 1: 1,950,000. Vertical scale 1: 6,500

the upper Elbe over the Oder to Stettin. These waterways have future possibilities, but only after improvement of the economic condition of eastern Germany. In conclusion, the plan by O. Taaks and Herzberg should be mentioned. This project proposes a connection of the Rhine to Emden after the construction of a canal from Wesel to Emden. However, it is hardly possible for this passage to replace the cheap Rhine waterway to Rotterdam.

10. The Merwede Canal of Holland

Holland, with its large network of canals, is very differently situated from other countries because of the ease with which its canals can be constructed. The extent to which inland navigation of Holland has developed is evident; for example, from the Merwede Canal which connects Amsterdam with the Lek and Waal (Gorinchem) Rivers; this canal carries transportation of approximately 15.3 million tons annually in 86,000 transport vessels. Ships up to 2,000-ton carrying capacity can navigate the Merwede Canal. The ships have a length of 80 m. (262 ft.), a breadth of 10 m. (33 ft.), and 2.4 m. (7.9 ft.) draft. The Rhine ships have attained a length of 123 m. (404 ft.), a breadth of 14.1 m. (46.2 ft.), and 2.85 m. (9.35 ft.) draft.[1]

Although the Merwede Canal serves Dutch transportation principally, very many of the ships also navigate the Rhine. The Merwede

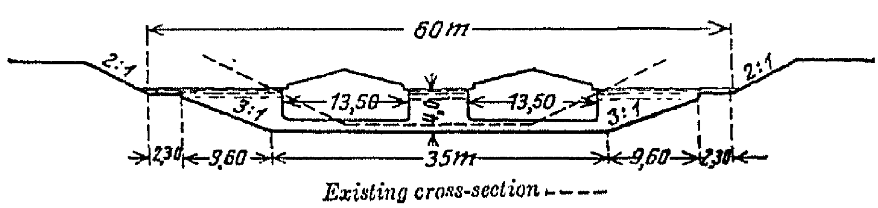


Fig. 565. Cross-section of the Merwede Canal

Canal has reached the limit of its capacity, particularly between Amsterdam and Vresswijk (Lek River). The channel is too narrow. The canal between Rotterdam, Antwerp, and other cities can be navigated with 3,500-ton ships. Two boats travelling in opposite directions on the present Merwede have difficulty in passing. The cross-section of the existing Merwede Canal is shown in Fig. 565 together with the newly planned cross-section. The present cross-section has a bottom breadth of 20 m. (66 ft.), 3.10 m. (10.2 ft.) depth under low-water, and 35 m. (115 ft.) surface breadth during low-water (determined 1892–93). These dimensions are similar to those of the newly constructed Midland Canal. It is of interest that, in spite of a ship breadth of only 10 m. (33 ft.), the difficulties when ships meet have been predominant. The bed breadth of the new cross-section formerly was 35 m. (115 ft.)

[1] *Bauing.*, Jan. 22, 1927, p. 55.

with a depth of 4 m. (13 ft.) below low-water. A berm is to be provided at both sides which will probably be covered by a growth of reeds to form shore protection. The theoretical surface breadth is 59 m. (194 ft.), including the berm the breadth is 60 m. (197 ft.). In Holland less emphasis is laid upon theoretical considerations, such as small resistance to navigation according to laboratory experiments, than upon good shore protection.

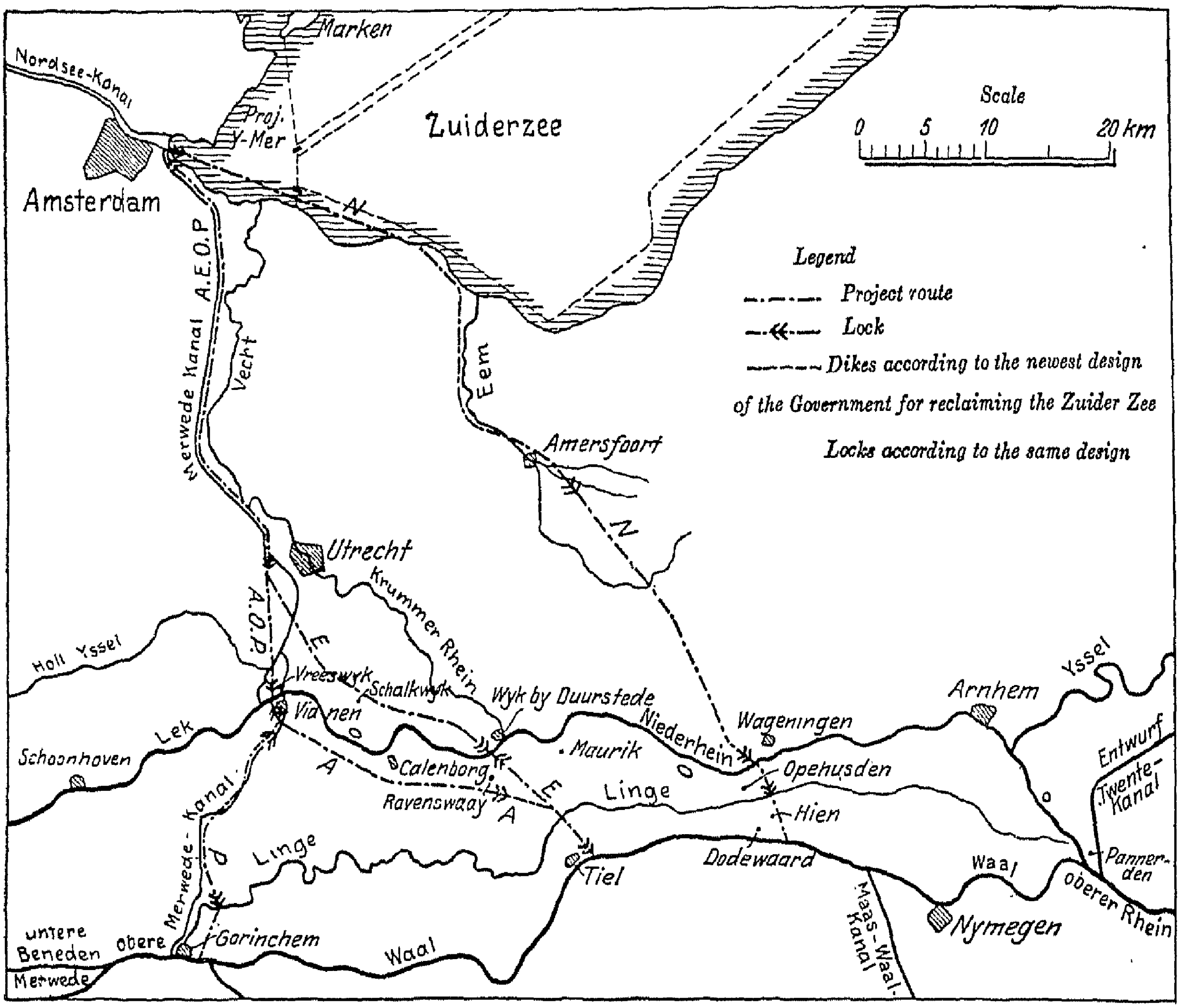


Fig. 566. Plan of the Merwede Canal

Five of the many new plans prepared are shown in Fig. 566. These have been investigated by a national commission. Table No. 23 summarizes the principal features of the designs. The Commission recommended Plan E as the most desirable of those listed in the table. Plan E proposes an excellent connection to the Lek River and also to the Waal and thereby to the upper Rhine. One of the advantages of Plan E is the short transportation time provided, especially to Ruhrort, the shorter river stretch, and the choice of a favorable crossing point of the lower Rhine. Low cost was not the deciding factor, because in this case it would have been necessary to develop the old Merwede Canal from

TABLE NO. 23

PLANS FOR THE IMPROVEMENT OF THE WATERWAY FROM AMSTERDAM TO THE RHINE

Design	Length in km.		Total length	Number of locks[1]	Average travel time in hours and minutes	Comparative cost in Reichsmarks according to unit cost from July to August, 1925	Remarks
	River passage	Canal passage					
A Amsterdam-Vreeswijk — Tiel-Pannerden	45	78	123	4	26–20	101,136,000[2]	The lock at Tiel is open for Waal stages below 0.37 m = MW (1901–1910)
E Amsterdam-Wijk at D-Tiel Pannerden	45	73	118	4	25–30	90,300,000[2]	The lock at Ravenwaaij is open for Lek stages under MW (1901–1910)
Na Amsterdam-Amersfoort-Hien-Pannerden (with use of the Ij Bay)	32	78	110	3	22–40	132,216,000	The same as above.
O Amsterdam-Vreeswijk (Regulated river at Pannerden)	84	47	131	1	23–00	50,349,600	
P Amsterdam-Vreeswijk-Gorinchem-Pannerden	86	69	155	4	31–30	81,984,000	The lock north of Linge may remain open about half of the year

[1] The lock at Zeeburg is left out of consideration for all cases since it is open nearly all the time.
[2] There is no allotment in these amounts for replacing the existing swing bridges by new ones with enlarged breadth and depth.

Amsterdam to Vreeswijk, and from there continue it over the Rhine. In spite of the lower Rhine crossing at Wijk, a longer canal route and shortening of the river stretch by use of the Waal were preferred to the former course.

It is of interest that with a canal length of 73 m. (240 ft.) (line E), a kilometer rate of about 1.25 million marks is incurred. The choice of a trapezoidal section corresponds to the view set forth in this treatise.

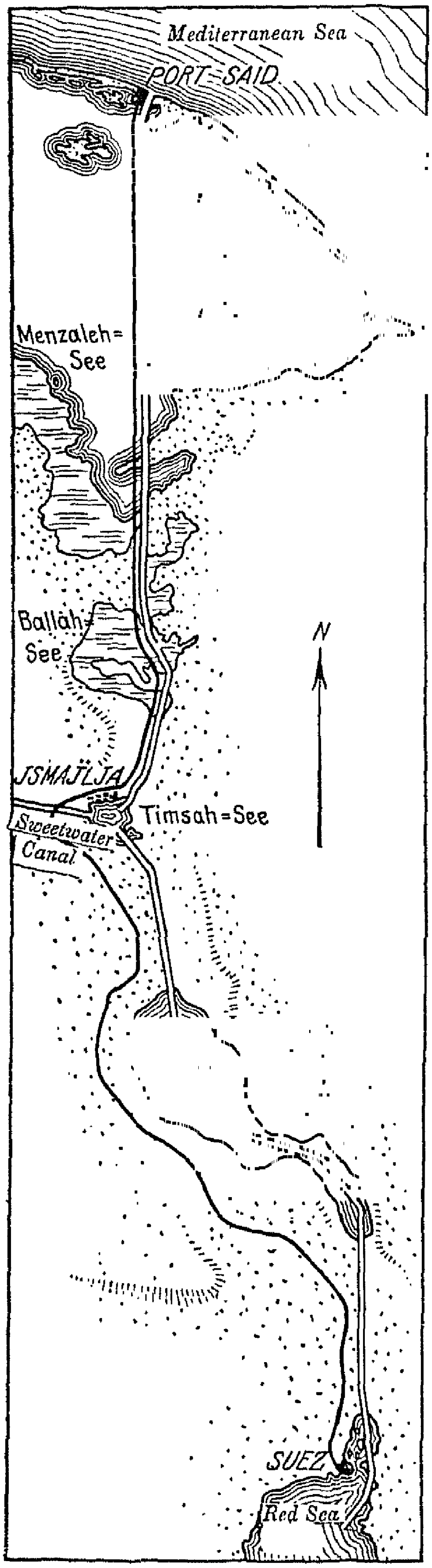


Fig. 567a. Layout of the Suez Canal

b. Sea Canals

1. Open Sea-Level Canals

a. Suez Canal

A waterway between the Mediterranean Sea and the Red Sea (Fig. 567a) was already constructed at the time of the Pharaohs. This consisted of a canal from the Nile to the Red Sea. Sethos I and Ramses II constructed the canal for the fleet in 1400 B.C. through Lake Timsah.[1] This canal became unusable, probably because of silting up as a result of a weak current toward the Red Sea. It was presumably to be newly constructed over a different course by Necho about 600 B.C., but after his death was abandoned by some 120,000 workers on the basis of an oracle, and was completed by Darius Hystaspes about 500 B.C. This canal failed, probably in a manner similar to the first, by becoming silted. Under Trajan a new canal was apparently constructed, which also must have failed, because General Amr reconstructed the canal under

[1] Since the gradient of the Nile from Cairo to the sea drops only 10 cm. (3.9 in.), the east arm, which was of importance in ancient times, was only about 1 m. (3.28 ft.) over mean sea level at a point northwest of Lake Timsah. The current was doubtless weak, and because of it the canal went to ruin in the course of time.

Omar in the seventh century A.D. from Cairo to the Red Sea. This one likewise was not usable a century later. Attempts of the Phoenicians and later of the Turks to rebuild the canal were not successful. Napoleon I similarly wanted to build a waterway, but his engineers reported that the level of the Red Sea was approximately 10 m. (33 ft.) higher than the Mediterranean Sea. This error was first corrected in 1841 by English officers. After this an Austrian engineer named Negrelli made accurate studies, and in Paris in 1856 proposed a design for the construction of the Suez Canal. Negrelli was awarded the contract but died in 1858. After this, the canal was completed by Lesseps who had bought up the stock. Construction was begun in 1859 and the canal opened for traffic in 1869. The importance of the Suez Canal lies in the fact that it shortens the course from Europe to India by about 5,000 sea miles (8,000 km.). The canal (Fig. 567a) has a length of 160 km. (99 mi.) beginning from the Mediterranean Sea at Port Said and ending in the Red Sea at Suez. A profile is given in Fig. 567b. The tidal variation at Port Said is usually .44 m. (1.44 ft.) and reaches .95 m. (3.12 ft.) at storm tide. On the Red Sea at Suez the usual tide is between .8 and 1.5 m. (2.6 and 4.9 ft.), storm tides reaching 3.24 m. (10.63 ft.). Accordingly, a weak current up to .6 m. (1.9 ft.) per sec. develops in the stretch from Port Said to Bitter Lake at Suez. These figures are true for the former bed breadth of 22 m. (72 ft.) but now, since the widening of the

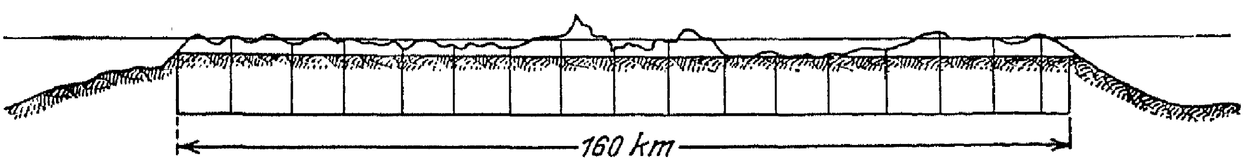


Fig. 567b. Profile of the Suez Canal

channel, the velocities have increased.[1] The variation of the cross-section and silting due to this current is negligible, and sanding up due to sand storms is of no importance. The choice of an open canal was thus very fortunate. In 1926 the transportation through the canal amounted to 26.1 million tons.

The deepening of the canal to 10 m. (33 ft.) was completed in 1924; the canal is to be deepened 2 m. (7 ft.) more. It was widened to 45 m. (148 ft.) bed breadth on the east side of the canal. Previously, recesses were allowed at 5 km. (3 mi.) interval on this side, the Asiatic side; the canal was widened by dredging the intermediate stretches. Flat side slopes of 1:3 to 1:4 were necessary because of the presence of easily moved sand along much of the stretch, but this did not play an important rôle in the cost inasmuch as the adjacent land was very low.

[1] Interesting computations by de Thierry with reference to tidal currents in canals (Inter. Navigation Congress in Chicago, 1911) showed agreement between computations and actual measurements.

Fig. 531 shows a cross-section of the canal, the old cross-section as well as the new one being indicated. The breadth is to be further increased to 65 m. (213 ft.) in the straight course and 80 m. (262 ft.) at curved stretches. The canal is supplied with fresh water by a special fresh water canal on the African side.

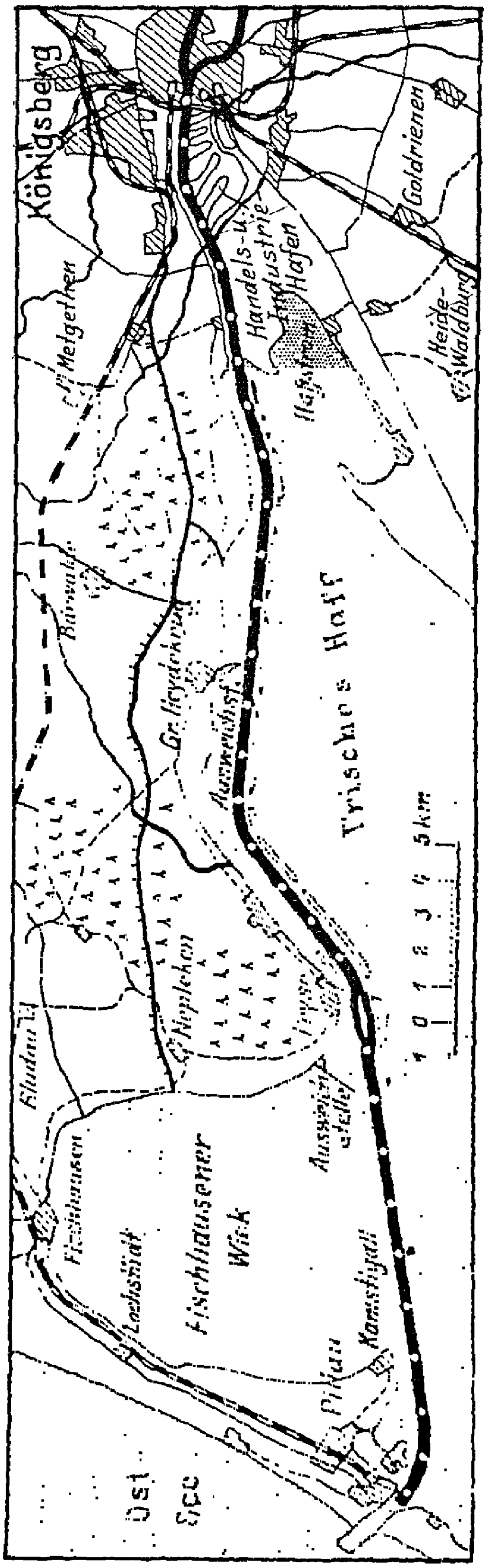


Fig. 568. Königsberg Sea Canal

β. *Königsberg Sea Canal*

The Königsberg Sea Canal became necessary because of the shallowness of the water in the Fresh Haff. It is constructed in part with a dike on one side and in part as an open channel. It is 42 km. (26 mi.) long and has a general width of 42.5 m. (139 ft.) at the bottom but at some locations widens to 90 m. (295 ft.) (Fischhauser Wiek). In sand the side slopes are 1:2.5, in silt 1:5. It was possible to dig the canal practically entirely by dredging. The depth originally (1901) amounted to 6.5 m. (21.3 ft.) but is now 9 m. (30 ft.) at mean water. The canal is of great importance to the city of Königsberg since it again raised this old sea city to the status of a practical sea harbor city. A plan view is shown in Fig. 568; a cross-section in Fig. 530.

γ. *Other Open Canals*

These include the canal of Corinth, the Chesapeake Canal, and the Cape Cod Canal.

2. Closed Sea-Level Canals (North-Baltic Sea)

An example of a closed sea-level channel is the Kaiser Wilhelm Canal which joins the North Sea to the Baltic Sea[1] (Fig. 569). The forerunner of the North-Baltic Sea Canal was the Eider Canal, constructed, 1777–84, to a depth of

[1] *Z. Bauw.*, 1896 to 1899, and *Zentralbl. Bauverw.*, 1907, p. 461.

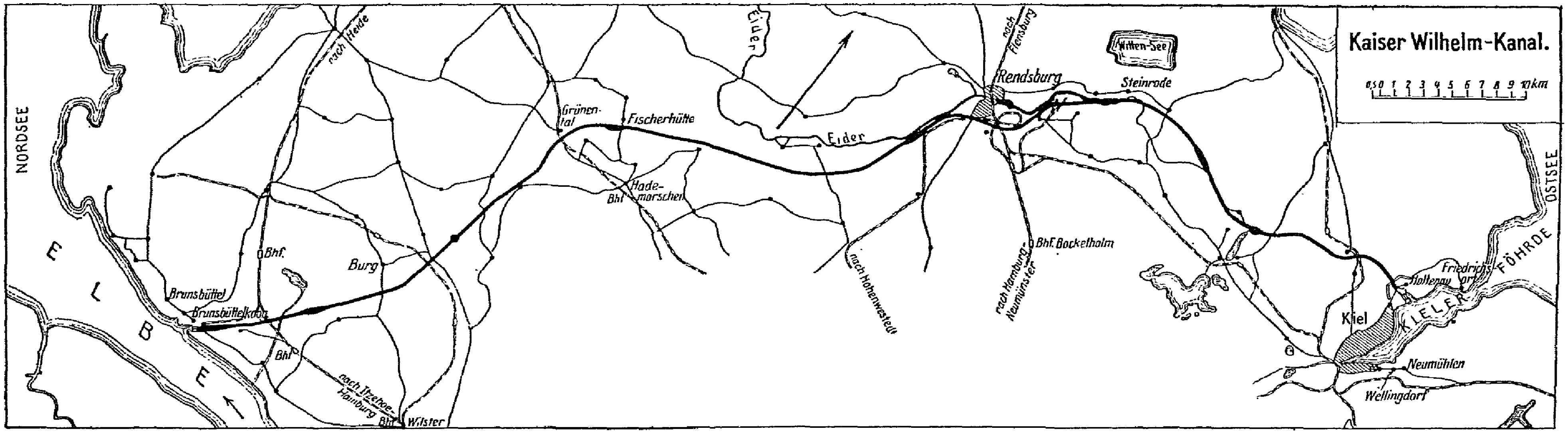


Fig. 569

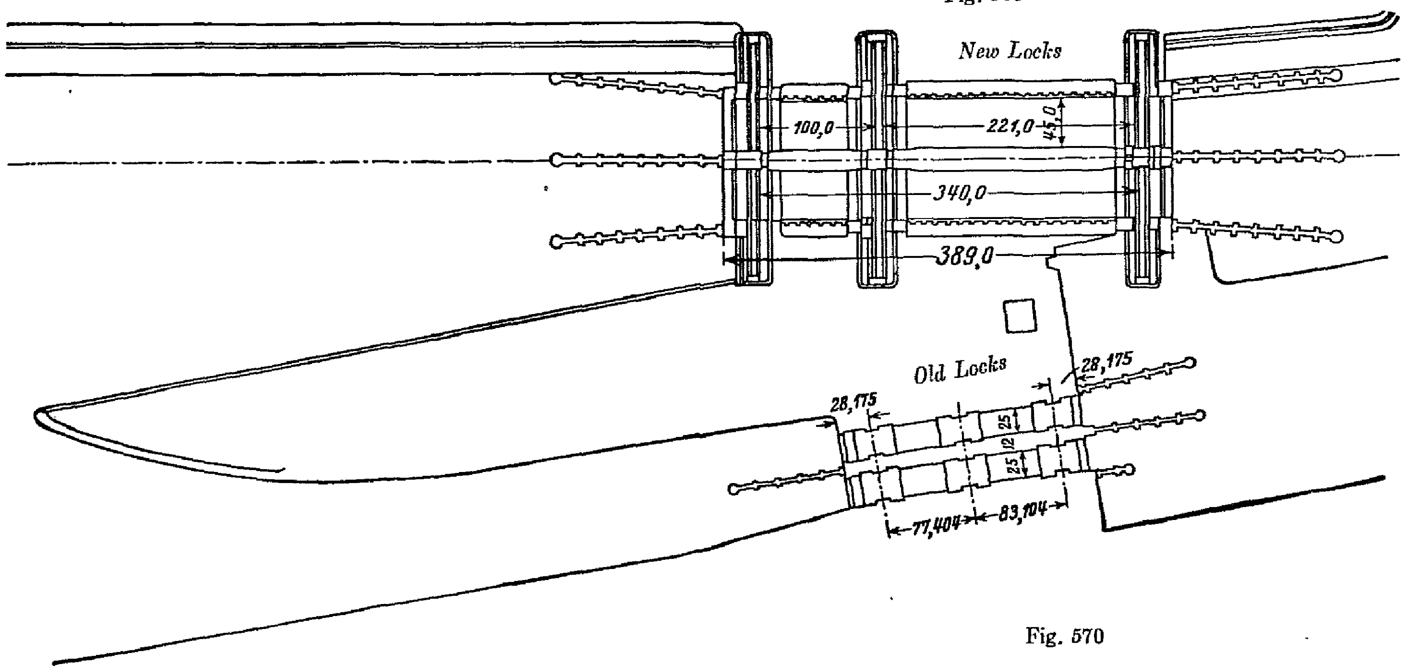


Fig. 570

Ostsee=Baltic Sea
Nordsee=North Sea

Fig. 569. Kaiser Wilhelm Canal. North-Baltic Canal

Fig. 570. Locks of the Kaiser Wilhelm Canal at Brunsbuettel

3 m. (10 ft.) which no longer exists. The North-Baltic Canal was constructed under direction of Fülscher, as a result of the agitation of Dahlström of Hamburg, at a cost of 156 million marks. The plans of Boden, 1887–1895, were used. It was enlarged under Hans W. Schulz (1907–14) with an expenditure of 220 million gold marks.

The design was developed principally for military purposes. The canal made it possible for the German war fleet to travel within a short time either to the North Sea or to the Baltic Sea, so that even though the entrance to the Baltic be blocked by an enemy fleet, separation of the two seas was not possible.

The economic significance of the canal lies, first, in shortening the route from the Baltic to the North Sea which, for example, amounts to 240 sea miles to London and 425 sea miles to Hamburg; second, in avoiding the journey around the dangerous Cape Skagen.

The canal begins at Holtenau in the Bay of Kiel and ends at Brunsbüttel on the Elbe, having a length of 98.65 km. (61.26 mi.). In spite of the somewhat unfavorable lay of the land, it was possible, by skilled location of the course, to construct the canal at the level of the Baltic Sea. It would not have been suitable, however, to construct it as an open canal; first, because large stretches of land on the North Sea side lie only 2 m. (7 ft.) under the present mean elevation of the canal water surface, and second, because the danger of silting up would have been very pronounced as a result of the daily inflow of water from the North Sea.

The usual tidal change on the North Sea side amounts to 2.8 m. (9.2 ft.); the greatest, 8.4 m. (27.6 ft.). The greatest difference in water level between the North Sea and the Baltic is practically 5 m. (16 ft.). These figures are significantly greater than the corresponding ones for the Suez Canal. The locks were of further advantage because with them the canal bottom could be laid higher than it could be if the low-water sometimes occurring in the North Sea would also develop in the canal. The old and the new cross-section of the channel is shown in Fig. 529. The canal provides a single passageway for large ships and a double passageway for ships of average dimensions. Originally, eight recesses were provided; the number at present is eleven. The former width was doubled in order to accommodate the present-day large vessels. A berm about 2 m. (about 7 ft.) under the water surface which was provided at both sides of the old channel was dispensed with in the new project because it did not prove to be permanent and caused obstructions to the bottom of the canal by caving. At the recesses the total breadth at the bottom is 134 m. (440 ft.) and 190 m. (623 ft.) at the surface. Four of these recesses were developed into turning basins

having a diameter of 300 m. (984 ft.) at the bottom and 340 m. (1,116 ft.) at the surface. The lengths of the recesses vary between 600 and 1,400 m. (between 1,969 and 4,593 ft.).

The sharpest bend now has a radius of curvature of 1,800 m. (5,904 ft.), the previous 1,000 m. (3,281 ft.) radius having proved to be too small. Eighty per cent of the bends have radii of 3,000 m. (9,843 ft.) or more. The canal was formerly closed off by two locks at each end, each of which was 150 m. (492 ft.) long, 25 m. (82 ft.) wide, and had a sill depth allowing ships of 8.5 m. (27.9 ft.) draft to pass at low water. The sill in Holtenau was approximately 9.8 m. (32.2 ft.) under middle water.

In addition to these older twin locks two new ones have been built at each end, each being 330 m. (1,082 ft.) long, 45 m. (148 ft.) wide, and 14.1 m. (46.3 ft.) sill depth below mean water (Fig. 570). The sill depth under usual low water on the Elbe is 14.42 m. (47.31 ft.), and the depth decreases to less than 12 m. (39 ft.) at the Elbe only about forty-two days each year during a short time of each tide. A sill depth of 12 m. (39 ft.) at low water is almost invariably present on the Baltic side. It is hoped thereby to provide passage for the largest ships of the future. The new locks were constructed in the dry by holding back the ground water. They are provided with roller pontoons as gates. All equipment is electrically operated.

Crossings for the streets and railways caused substantially greater difficulty than in the construction of the Suez Canal. Some of them were developed as elevated bridges, the bottom edges of which are 42 m. (138 ft.) over the highest canal water stage, others as swing bridges, still others as pontoon bridges and ferries. After the widening, all railroads will cross the waterway on elevated bridges. Both shores of the canal are illuminated at night by a line of 25 candle-power incandescent lamps.

The traffic on the canal amounting to 18.2 million tons annually is only slightly less than that of the Panama Canal.

3. Sea Canals with Variously Elevated Pools

a. The Manchester Sea Canal

The Manchester Canal was constructed to join the large manufacturing city of Manchester with the sea and to make it independent of the Liverpool harbor. It is noteworthy that between the two cities there already existed five railroad lines and two inland navigation canals. Manchester lies on the Irwell River, a tributary of the Mersey (Fig. 571).

The tidal change at Liverpool during spring tide is 8 m. (29 ft.). Since Manchester lies about 21 m. (69 ft.) above mean sea level, a sea-

level canal was not possible, particularly since it was desired to have the harbor in the city. The canal was constructed in five pools, the outer-

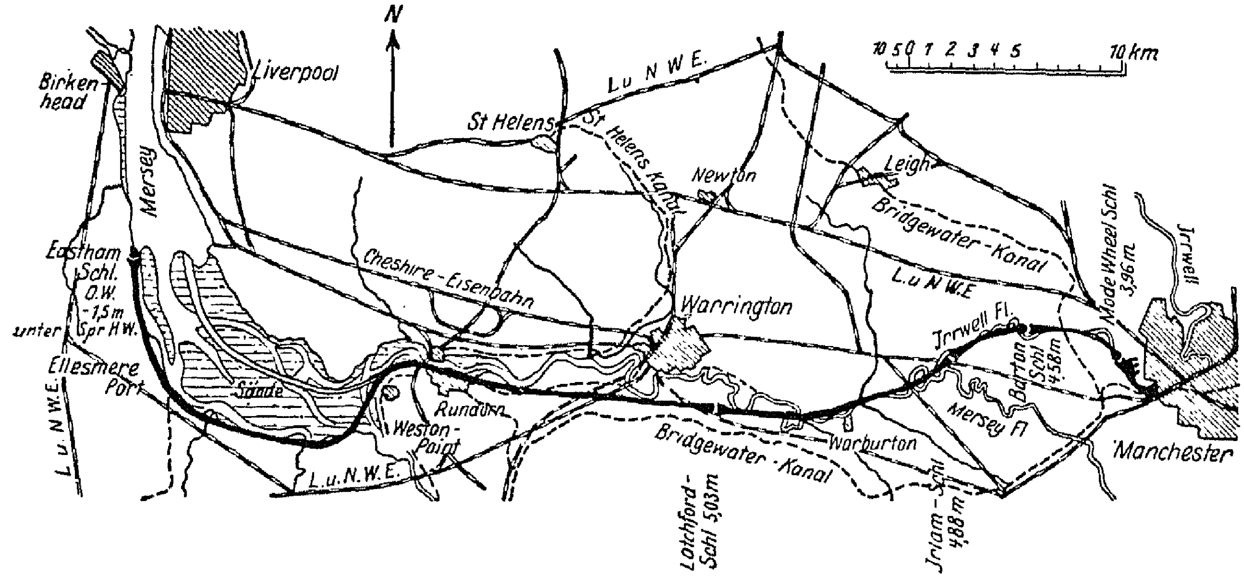


Fig. 571. Manchester Sea Canal

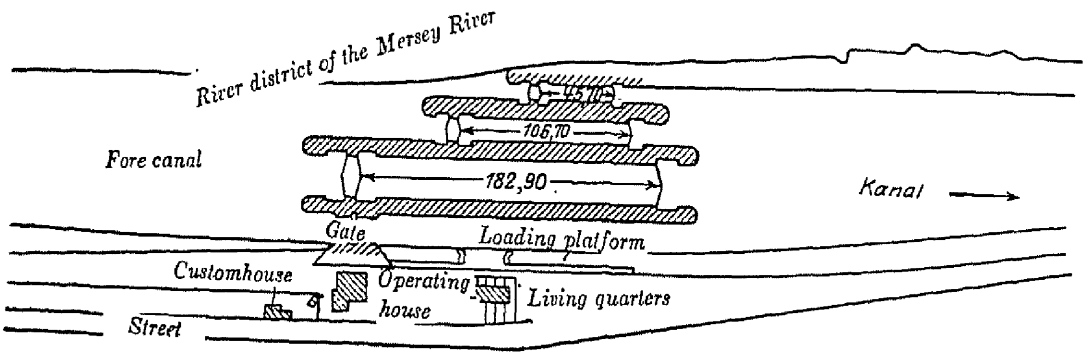


Fig. 572. Group of locks of the Manchester Sea Canal

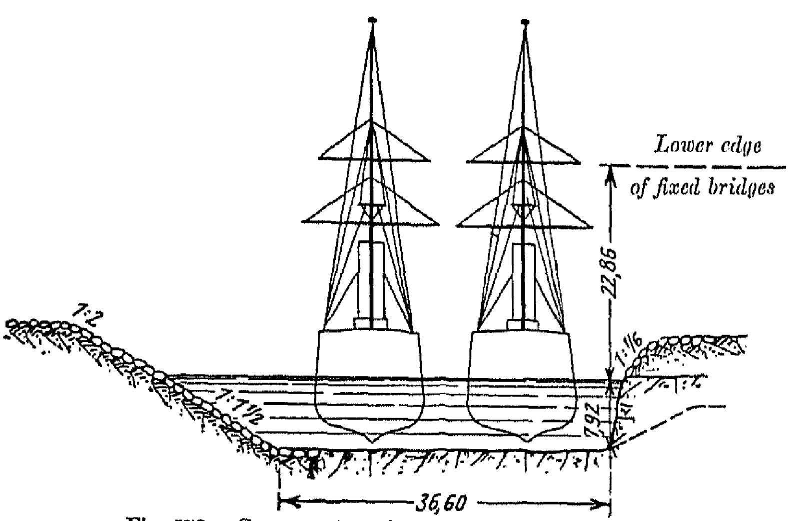
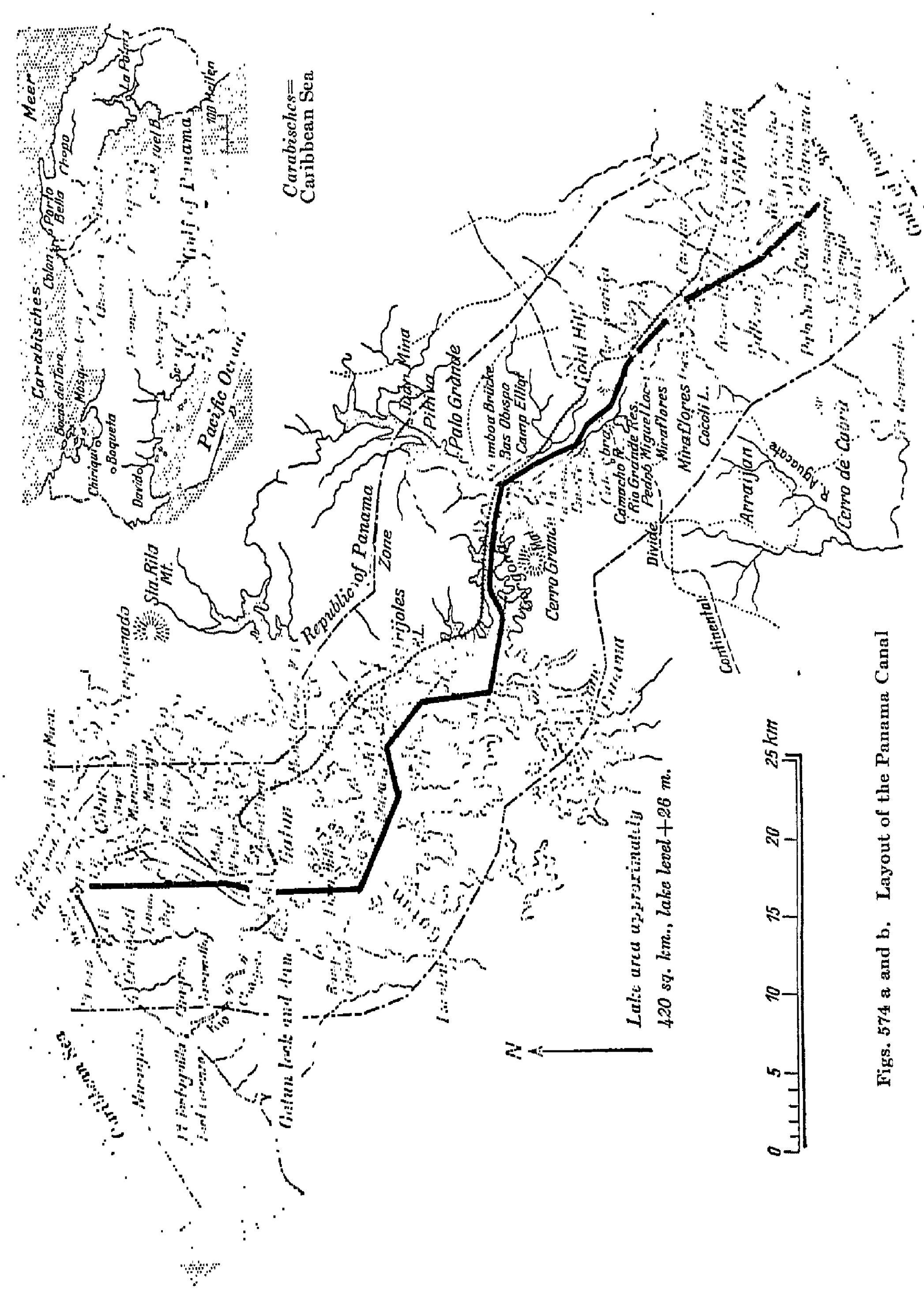


Fig. 573. Cross-section of the Manchester Sea Canal

most of which is 1.5 m. (4.9 ft.) below the crest of the spring tide. The other pools have a drop of at least 4 m. (13 ft.) and a maximum of 5 m.

(16 ft.). Double locks are provided at each change in level and three locks were constructed at the entrance to the Mersey, the latter layout



Figs. 574 a and b. Layout of the Panama Canal

being shown in Fig. 572. The breadths of the locks are respectively 24.38 m. (80.0 ft.), 15.24 m. (50.0 ft.), and 9.14 m. (30.0 ft.), and although the normal water depth of the canal has hitherto been only

8 m. (26 ft.), the locks are constructed with a sill depth of 8.32 m. (27.30 ft.) so that further deepening of the canal is possible.

The canal has a minimum radius of curvature of 2,000 m. (6,562 ft.). It was constructed as a two-ship canal for all ships which could pass its locks. Nevertheless, since its bottom breadth is only 36.6 m. (120.1 ft.), it is now to be considered as a one-ship canal because newer ships of about 23 m. (75 ft.) breadth and full cross-section can still pass through the locks but can not pass each other in the channel. A cross-section in which the left shore is earth and the right bed rock is shown in Fig. 573. Interesting features of this canal are the street and inland canal crossings. For example, the Bridgewater Canal crosses this sea way through a swing bridge.

β. Panama Canal[1]

The first attempt to pierce the Isthmus of Panama, which joins the continents of North and South America and separates the Atlantic and Pacific oceans, was undertaken by the builder of the Suez Canal, Lesseps. The mean tidal change at Colon on the Atlantic Ocean amounts to .3 m. (.98 ft.), at Panama on the Pacific it reaches 4 m. (13 ft.). The maximum tidal change at Colon and Panama respectively is .75 m. (2.46 ft.) and 6.8 m. (22.3 ft.). After a construction expenditure of 800 million marks, Lesseps had to give up the work because of the lack of funds.

A new organization was formed from the old one and this was taken over by the United States in the year 1903. In 1905 an international consulting commission was called together to determine whether the canal should be a sea-level canal, as originally planned, or a lock canal with several levels. The majority, principally Europeans, recommended the sea-level canal; the minority the lock canal. The United States Government concluded to accept the recommendation of the minority of the Commission, so the canal was constructed as a lock canal with three steps.

The canal (Figs. 574–576) begins on the Atlantic side with an open approach canal 11 km. (6.8 mi.) long, then ascends in a three-step flight of twin locks at Gatun to the artificially created Gatun Lake, lying 25.8 m. (84.6 ft.) above mean sea level, passes through the lake and the adjoining Culebra Cut for a distance of 52 km. (32 mi.), rises at the Pedro-Miguel twin locks to an intermediate stretch 2 km. (1.24 mi.) long (the Miraflores Lake) and by means of the two step Mira-

[1] Fülscher, "Panamakanal," *Z. Bauw.*, 1907.— Tincauzer, "Panamakanal," *Z. Bauw.*, 1911.— O. Franzius, "Der Sookanal und der Panamakanal," *Z.V.d.I.*, 1912.— O. Franzius, "Panamakanal," *Z.V.d.I.*, 1914.

flores twin locks the course drops to the level of the Pacific Ocean which is reached by an approach canal 13 km. (8 mi.) long. Including locks, the canal is approximately 80 km. (49 mi.) long. Fig. 576a and Fig. 531 show diagrammatically the existing cross-section. Figs. 576 a and c indicate the narrowest and broadest locations proposed in the sea-level layout. A deciding factor in the choice was that the lock canal could be constructed in about two-thirds of the time and for about two-thirds of the cost required for a sea-level canal, while at the same time passage through the locks was considered less dangerous than the approach of two ships on the long narrow stretch of the sea-level design. In the latter, the narrow stretch would have been about 21 km. (13 mi.) long with a bottom width of 45.75 m. (150 ft.) while the narrowest stretch of the lock canal is approximately 8 km. (5 mi.) long with a bottom width of 91.5 m. (300

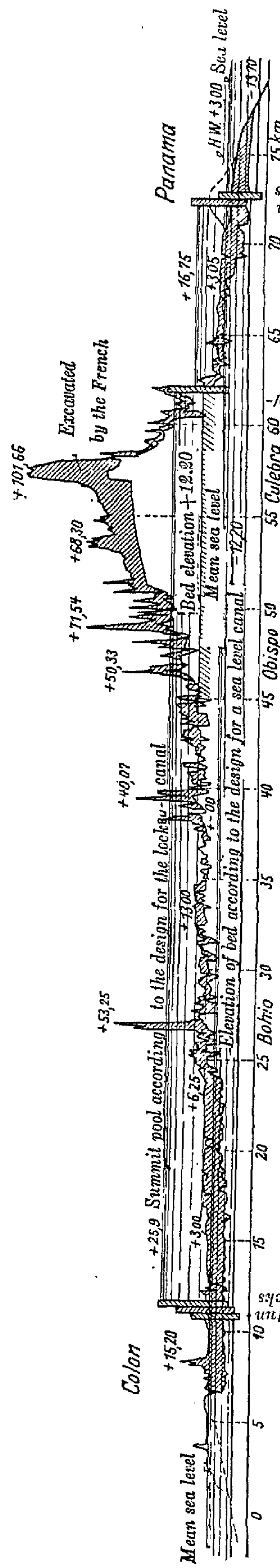


Fig. 575. Profile of the Panama Canal

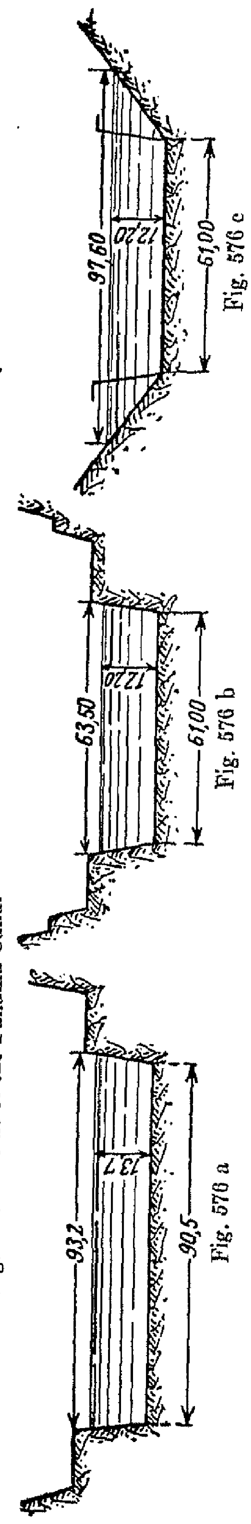


Fig. 576a. Narrowest section of completed canal. Side slopes in earth stretches 1: 1.5, in lock 10:1
Figs. 576 b and c. Broadest section of the plan which was proposed for a sea level canal

ft.). Over a distance of 31 km. (19 mi.) the lock canal traverses artificially created lakes having a navigable breadth of 305 m. (1,000 ft.) while the sea-level canal would at no place have been wider than 61 m. (200 ft.) at the bottom.

The lock canal received a depth of 13.7 m. (45.0 ft.) throughout, which even in weakest rain periods does not sink below 12.7 m. (41.7 ft.) in the summit pool, while its competitor would have been made only 12.2 m. (40.0 ft.) deep. The time necessary to pass the lock canal is also less, and in spite of this the development cost several hundred million marks less than the sea-level project would have cost. Nevertheless, the size of the locks is conspicuous as they are only 33.5 m. (110 ft.) wide, 304 m. (1,000 ft.) long, and 12.2 m. (40 ft.) minimum depth at the port sill. It is possible that the breadth will become inadequate in the distant future. (Compare with North-Baltic Sea Canal.)

The time necessary to construct the lock canal was nine years and was computed to take 13 years for the sea-level plan. The earth and rock movement had the greatest influence on the construction period. The side slopes of the deep cuts consist of bed rock but contain thin clay layers. Consequently, after tropical cloud-bursts, serious caves occurred, necessitating flattening the slopes and excavating more rock. In some places the slides were so serious that some doubt was beginning to arise concerning the possibility of constructing the canal. In 1911 during the principal construction period, for example, the author observed a landslide which buried several large steam shovels; on the sliding mass stood a house and several undamaged palms. In any case, because of the serious caves, it is not certain that the construction of a sea-level canal, which would have required a 26 m. (85 ft.) increase in depth, would have been at all possible.

Another objection to the lock canal was the danger of earthquake. Notorious earthquake regions still exist north and south of the isthmus. Proponents of the sea-level canal feared the breaking down of the large locks under the influence of earthquake. In the city of Panama a flat arch, which has existed for centuries, appears to be proof that the earthquake dangers are not great. It is likely that in the canal zone there will be a vibration of the ground rather than a real earthquake. To the present no disturbances have developed. This, of course, is no proof of the absence of danger.

The last point of importance was the danger of lack of water-tightness of Gatun Lake or of a break in the Gatun Dam. It was feared that the bed rock might contain fissures which were covered only at the surface. However, experience proved the lake bottom to be water-tight. The Gatun Dam is constructed to dimensions making it possibly the safest dam that has ever been built. Lake Miraflores became salty so quickly by the rise of salt water in the locks that water can no longer be taken from this lake for drinking to serve the city of Miraflores.[1] The water of Gatun Lake doubtless also becomes brackish, but to a less noticeable degree because of the large inflow of fresh water.

Practically without exception, the facts have shown the erectors to be correct in their choice. The canal was opened January 1, 1915.

[1] *D.A.Z.*, of August 3, 1914. Supplement, "Kraft und Stoff."

Transportation over it has grown to the extent that it exceeds the amount passing in the Suez Canal. There is now some thought of the necessity of constructing a second canal (possibly through Nicaragua).

The cost of the lock canal was about 1,600 million gold marks. The Panama Canal is in strong competition with the American railways and has forced large freight reductions in the east-west continental transportation. In 1926 the canal was passed by 19.8 million tons of traffic.

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